CHAPTER 3
TENSIONED ROCK BOLTS

3-1. General.

a. The use of tensioned reinforcement elements is included in most rock reinforcement systems. The desired result of a tensioned rock bolt installation is a permanently tensioned reinforcement element with positive bond to the rock. Basic areas of concern in rock bolt installation are:

(1) Obtaining anchorage.

(2) Tensioning to the desired prestress, thus placing the rock around the bolt in compression.

(3) Locking the prestress in the bolt.

(4) Protecting against loss of anchorage and corrosion and utilizing the shear strength of the bolt.

b. Each area of concern is critical to a good permanent reinforcement system. A number of hardware types, techniques, and bonding materials have been used to achieve the desired installation. Some methods are more common and therefore of more interest than others. The methods and hardware types in common use are described in this section. The effectiveness of these methods in achieving a good final installation is discussed.

c. Selection of specific rock bolt hardware, grouting material, and installation method is often the result of personal experience of the design engineer as well as the result of cost studies. Almost any well planned and tested procedure will produce the desired installation. However, even the most highly proven techniques may fail to give satisfactory results if careful attention to detail is not practiced during installation.

3-2. Anchorage Methods. Adequate anchorage is critical to the proper performance of the reinforcement system. It is most critical between the time of initial tensioning and the time of full length grouting. During this period any creep or slippage of the anchor negates a portion of the reinforcement potential of the bolt. Total failure of the anchorage may have costly if not disastrous consequences. Until a bolt has been grouted full length and the grout has set up, the anchorage determines the percentage of total bolt strength that is available to reinforce the rock against discontinuous movements. It is therefore

desirable to achieve anchorage strength in excess of the ultimate strength of the bolt. Anchorage strength may be limited by the anchor itself or the rock type and quality. There are two types of anchorage in general use. These are mechanical anchorages and grouted end anchorages. The mechanical types make use of an expanding element that is forced against the walls of the borehole to deform the rock and to provide frictional resistance to pullout. Grouted end anchorages rely on a bonding medium between a portion of the reinforcing element and the rock to develop the desired anchorage strength. Combinations of grouted end and mechanical anchorages are also possible but are not generally used because installation techniques are more complicated and time consuming. However, slot and wedge bolts have been installed in downholes following the placement of grout in the bottom of the hole. Regardless of the anchorage type used, subsequent full length grouting after tensioning improves the reinforcement capability of the element and ensures permanence. In the following paragraphs, the various anchorage types are discussed.

    a. Slot and Wedge Anchorage. With the slot and wedge type, anchorage is obtained by inserting the wedge into the slotted end of the bolt and expanding the slot by driving the wedge against the end of the drill hole. Bolts with wedge-type anchorage hold best in hard, sound rock. To assure good anchorage, the hole length must be accurately drilled to within 3 inches and heavy driving equipment is needed. The use of slot and wedge bolts was once very common, but with development of expansion shell anchorage their use has declined rapidly and they are now seldom used in civil engineering projects. A slot and wedge anchorage is illustrated in figure 3-1. This type of bolt can be rapidly fabricated on the job and used in an emergency.

    b. Expansion Anchorages.

    (1) The expansion-type anchorage device obtains its anchorage by the action of a wedge or cone moving against a shell (or fingers) and expanding the shell against the sides of the hole. Application of torque to the bolt moves the threaded wedge or cone forcing the shell against the rock. With headed bolts the anchor expansion and tensioning can be accomplished in one operation, provided enough thread remains after the initial anchor expansion is completed to bring the bolt under full design tension. Most types of expansion anchorages are patented and have variations in diameter, length, and serrations.

    (2) Many expansion units are made of a cylindrical shell into which a tapered plug is drawn, as shown in figure 3-2. A support nut or upset ears on the rod are required for installation. This type of expansion unit usually has four faces and is used in soft rock. At the present time, these anchors are made and successfully used only for rock
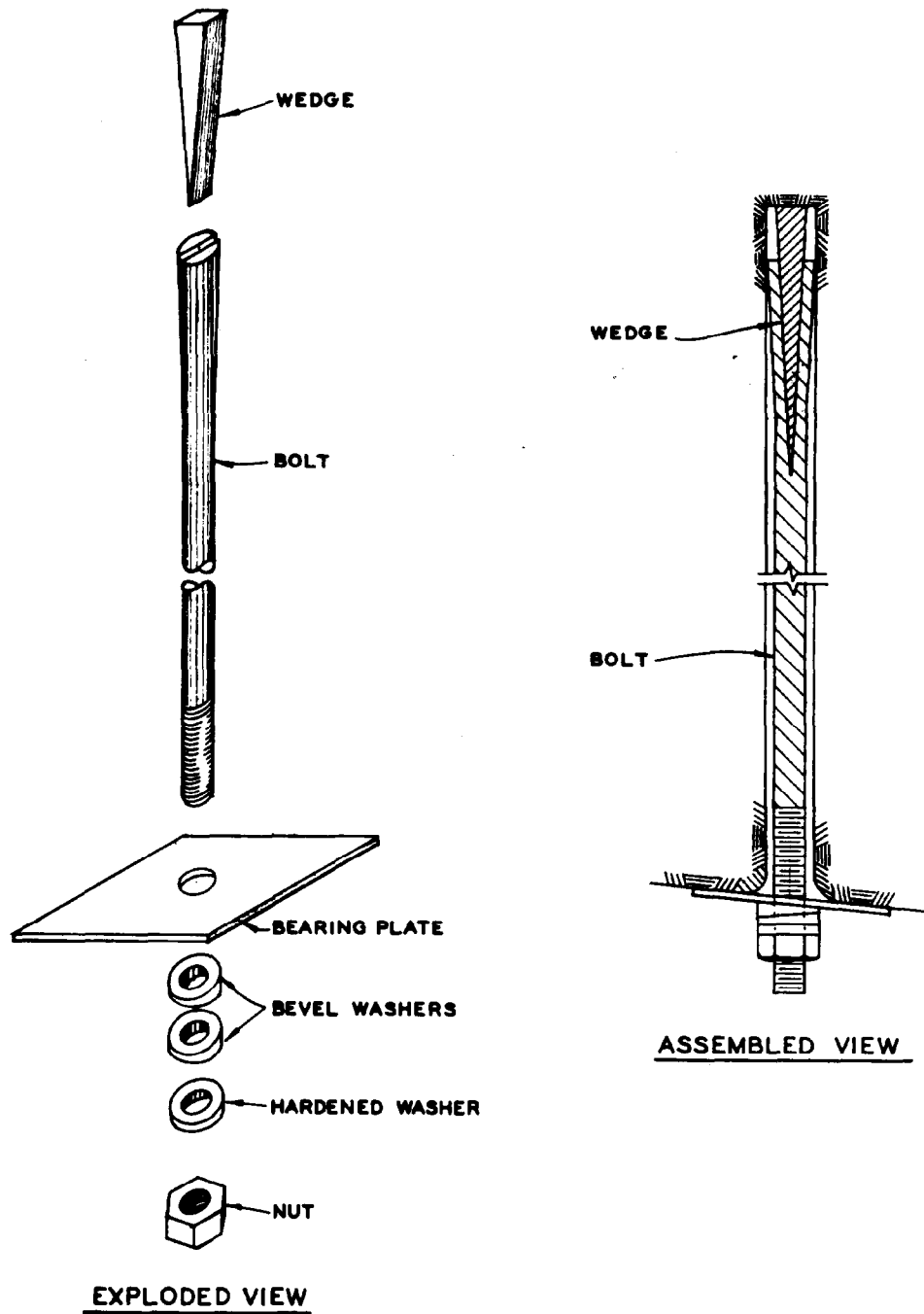
WEDGE

BOLT

WEDGE

BOLT

BEARING PLATE

BEVEL WASHERS

HARDENED WASHER

NUT

ASSEMBLED VIEW

EXPLODED VIEW

Figure 3-1. Slot and wedge rock bolt.

3-3

WEDGE

EXPANSION
SHELL

SUPPORT
NUT

BOLT WITH
SQUARE HEAD
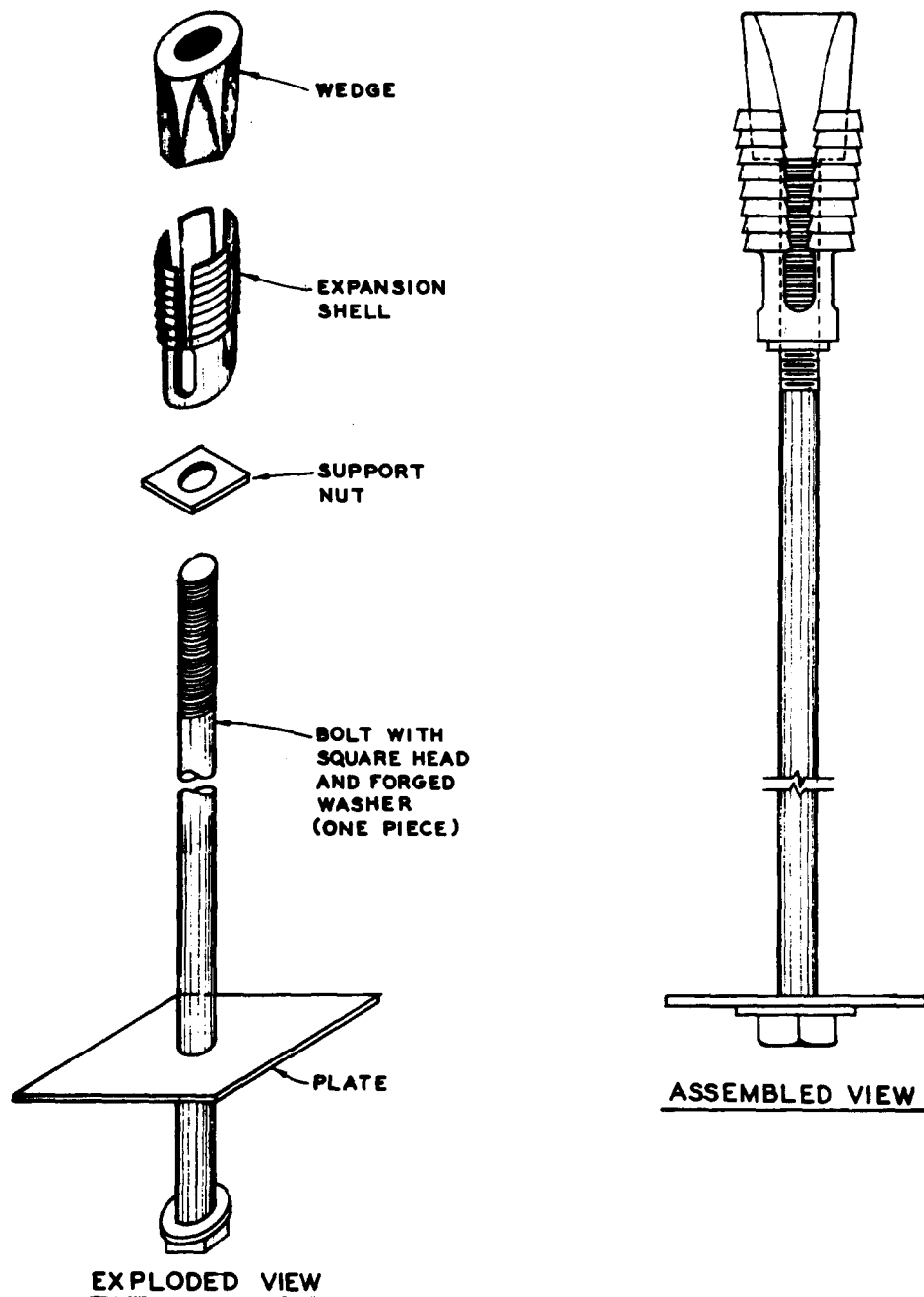AND FORGED
WASHER
(ONE PIECE)

PLATE

EXPLODED VIEW

ASSEMBLED VIEW

Figure 3-2. Regular expansion anchorage--headed bolt.

bolt diameters of 5/8 inch or 3/4 inch and are used primarily in mines rather than in civil engineering works.

(3) Bail type expansion units are made with a bail or strap between the leaves of the shell for support during installation (figures 3-3 and 3-4). During insertion of the bolt and torquing to set the anchor, the bail keeps the leaves at the same position on each side of the rod as the wedge is moved. Bail types can also be expanded directly (without torquing the bolt) if the bolt is tensioned by direct pull. These expansion units have two or four faces and are used in hard rock. These units, at the present time, are successfully used on 5/8-inch to 1-1/2-inch-diameter rock bolts.

(4) Another expansion shell type unit has a cylinder that is slotted on one side and expands as the cone-shaped wedge is moved toward a thrust collar (figure 3-5). These units have been successfully used on 5/8-inch to 2-inch-diameter rock bolts. By changing the length of the cylinder and plug these bolts can be used in moderately soft to hard rock.

(5) Expansion type anchorages can now be obtained which have a tandem or twin anchor system. Both anchors are set by the same operation. The use of this type of anchor has been limited to a few specific cases.

(6) For use primarily in softer rocks, a special one-piece expansion shell is available which requires a specially reamed conical cavity at the anchor end of the drill hole. A steel bar threads directly into the bottom of the shell. The upper portion of the shell is made of spring steel split into eight "leaves" held in a collapsed position by a special fitting. Once installed in the hole, the "leaves" are released to expand into the cavity by impacting the special fitting against the back of the hole. The rock compressive strength is thus utilized to develop anchorage rather than the friction force developed between the shell and the rock as is the case with other expansion shells.

(7) Table 3-1 lists sizes of commercially available mechanical anchorages for rock bolts.

   c. <u>Grouted End Anchorage</u>.

(1) With a grouted end anchorage, the length of element embedment varies with the type and condition of the rock and the bonding medium used. Portland cement, gypsum, and chemical grouts or mortars have been used successfully. The required embedment length in a given rock must

BAIL

EXPANSION SHELL

WEDGE

BOLT

BOLT

BEARING PLATE

BEVEL WASHERS

HARDENED WASHER

NUT

ASSEMBLED VIEW

EXPLODED VIEW
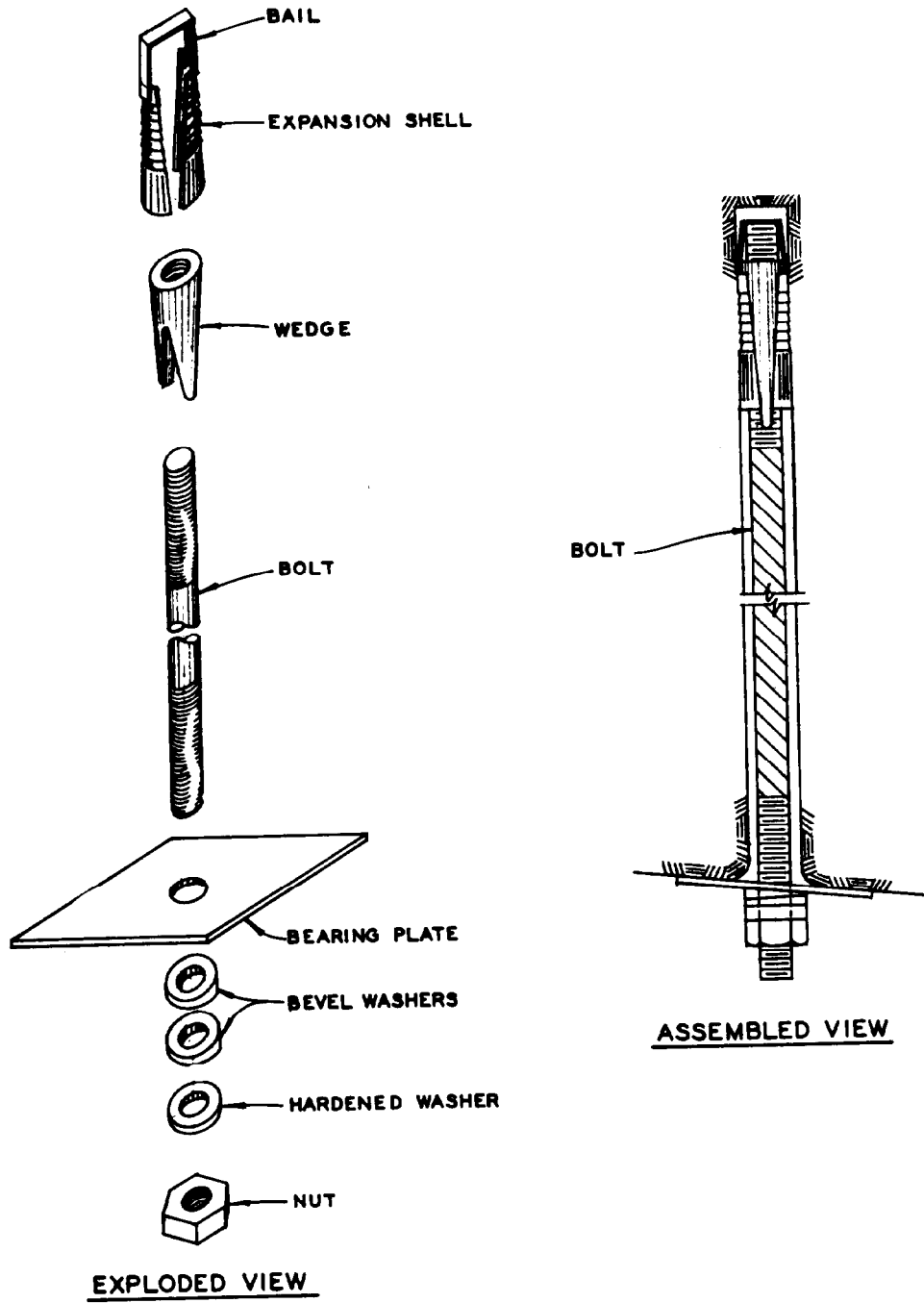
Figure 3-3. Bail expansion anchorage--solid bolt.

EXPANSION SHELL

WEDGE

GROUT/VENT TUBE

SMOOTH BOLT

BEARING PLATE

BEVEL WASHERS

HARDENED WASHER

NUT

QUICK SET
MORTAR SEAL
AND PAD

SHORT GROUT
TUBE

ASSEMBLED VIEW

EXPLODED VIEW
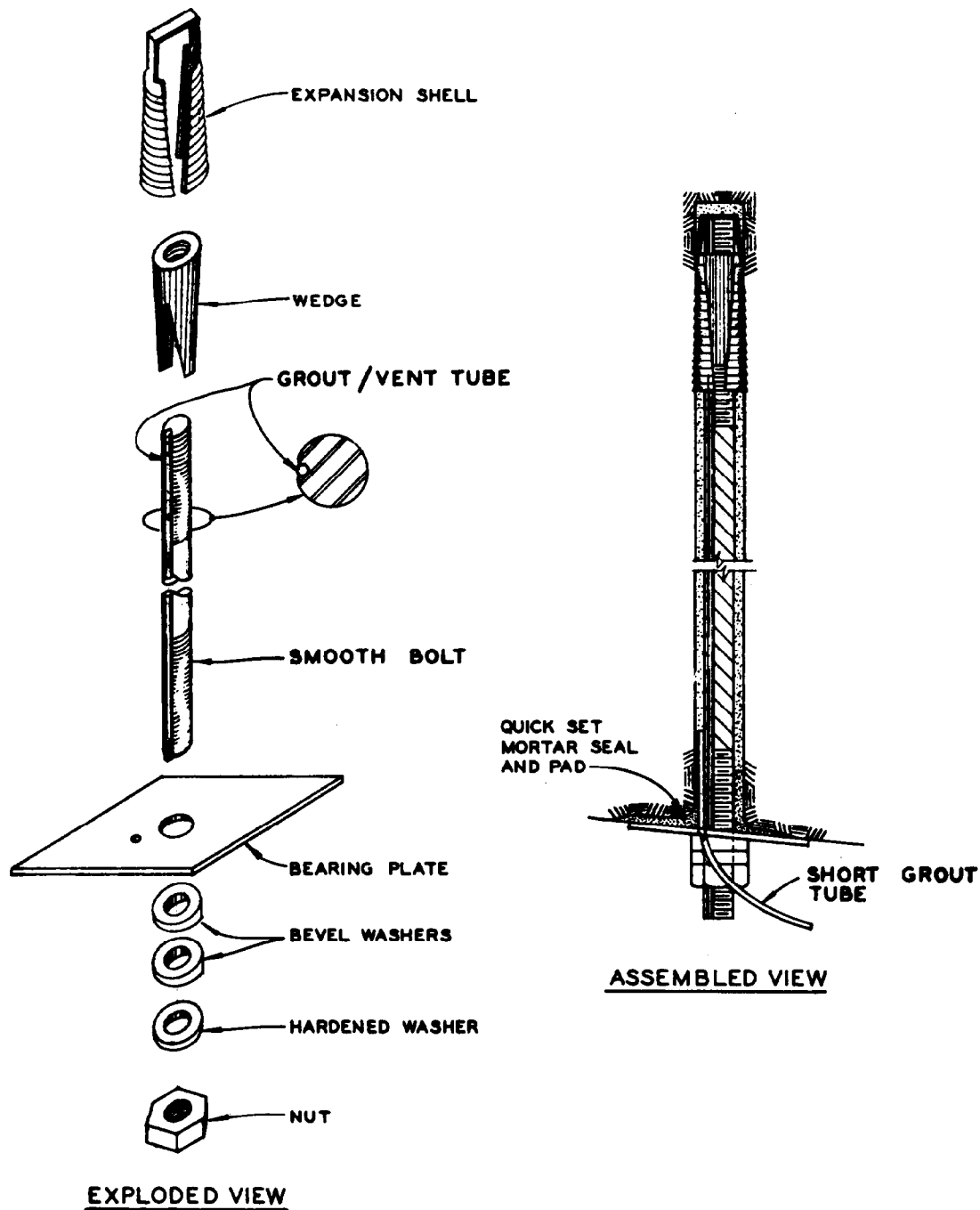
Figure 3-4. Groutable smooth bar rock bolt
with integral grout tube.

TAPERED PLUG

EXPANSION SHELL OR CONE

THRUST COLLAR

DEFORMED BOLT WITH HOLLOW CORE

BEARING PLATE WITH KEYHOLE SLOT

BEVEL WASHERS

HARDENED WASHER

NUT

EXPLODED VIEW

QUICK SET MORTAR SEAL AND PAD
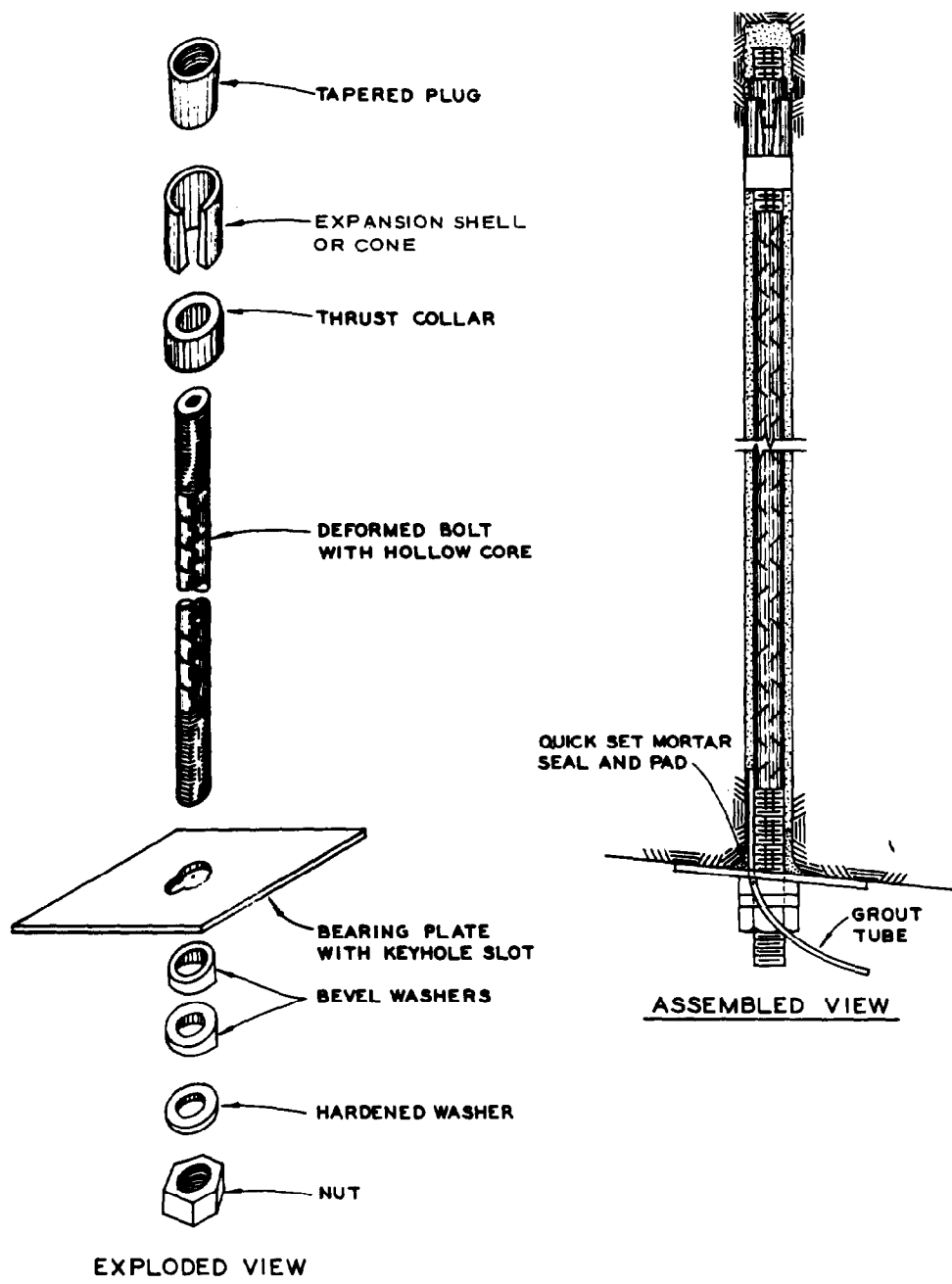
GROUT TUBE

ASSEMBLED VIEW

Figure 3-5. Hollow groutable deformed bar rock bolt.

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Table 3-1. Examples of a Number of Commercially Available
Rock Bolts with Mechanical Anchorage

| Type of Anchorage | Bolt Sizes Available | | | Remarks |
| | Smooth Bar Headed Bolts, in. | Threaded Bolts with Nuts | | |
		Solid Bar, in.	Groutable	
Expansion shell	--	1/2 - 2 in 1/8 increments	No. 8, No. 11, No. 16 (hollow, deformed)	Specializes in rock bolts for use in civil engineering works
Expansion shell, "Pattin"	5/8 - 1	5/8 - 1	--	
Slot and wedge	--	1	--	
Expansion shell	5/8 - 1	--	--	
Slot and wedge	--	1-2-1/2		
Expansion shell	5/8 and 3/4	--	--	
Slot and wedge	--	1	--	
Expansion shell	5/8 and 3/4	1	No. 8 (hollow, deformed)	
Slot and wedge	--	1 - 2-1/4	--	
Expansion shell	--	--	1-in. (smooth bar)	Manufactured with grout/vent tube installed in groove along bolt
Expansion shell for 5/8-in.- and 3/4-in.-diam bolts	--	--	--	Supplies shells only
"Cone" expansion shell	--	--	--	For use in soft rock with 3/4-in.- and 1-in.-diam bolts

be determined by conducting pull tests on the proposed installation. A grouted end anchorage is suitable for use in almost any rock type and holds well even in badly fractured rock. However, with portland cement and accelerators or with some of the chemical grouts, a waiting period of four to eight hours is required before sufficient strength is developed to allow tensioning of the element. Polyester grouts are available to develop sufficient strength within 5 minutes to 30 minutes. With gypsum grout, a waiting period of approximately 30 minutes is required.

(2) The ultimate strength of commonly used reinforcing elements installed in down-holes can be developed by simply embedding the lower end of the element in grout placed by gravity flow at the bottom of the hole. For up-holes, special techniques and aids have been developed to keep the grout at the upper end of the hole. Several processes (some patented) using perforated sleeves, prepackaged resin cartridges, grout transfer tubes, and pressure grouting with pumps are being used successfully. These are now also used in many down-hole installations.

(3) One type of grouted end anchorage utilizing grout tubes through which grout is pumped is shown in figure 3-6. Liquid cementations (portland cement or gypsum) and chemical grouts have been injected under pressure by this method. Where open joints or fractures exist near the anchor area, grout will tend to fill the fractures to help consolidate and strengthen the rock. In up-holes, grout is injected through the shorter tube with the longer tube serving as the air exhaust vent. In down-holes, the function of the tubes is reversed. A return of grout through the air exhaust tube indicates that the anchorage area is completely filled.

(4) Another grouted anchorage system (figure 3-7) uses perforated half-sleeves to retain mortar at the desired location. The half-sleeves are packed with mortar, tied together, and the bar inserted through the sleeve to extrude the mortar through the perforations and completely fill the anchorage area.

(5) The most recent developments in grouted anchorages have been in connection with the formulation and packaging of polyester resin grouts which develop ultimate element strengths within minutes of installation, figures 3-8 and 3-9. If properly installed, these systems incorporate most of the characteristics considered desirable for rock reinforcement. Somewhat longer length bars are employed with this system to obtain anchorage. However, they are particularly useful in weak rocks, or highly fractured zones.

DEFORMED RE-BAR

GROUT TUBES

FOAM RUBBER SEAL

STEEL COLLAR

PLATE

BEVEL WASHERS

HARDENED WASHERS

NUT

EXPLODED VIEW
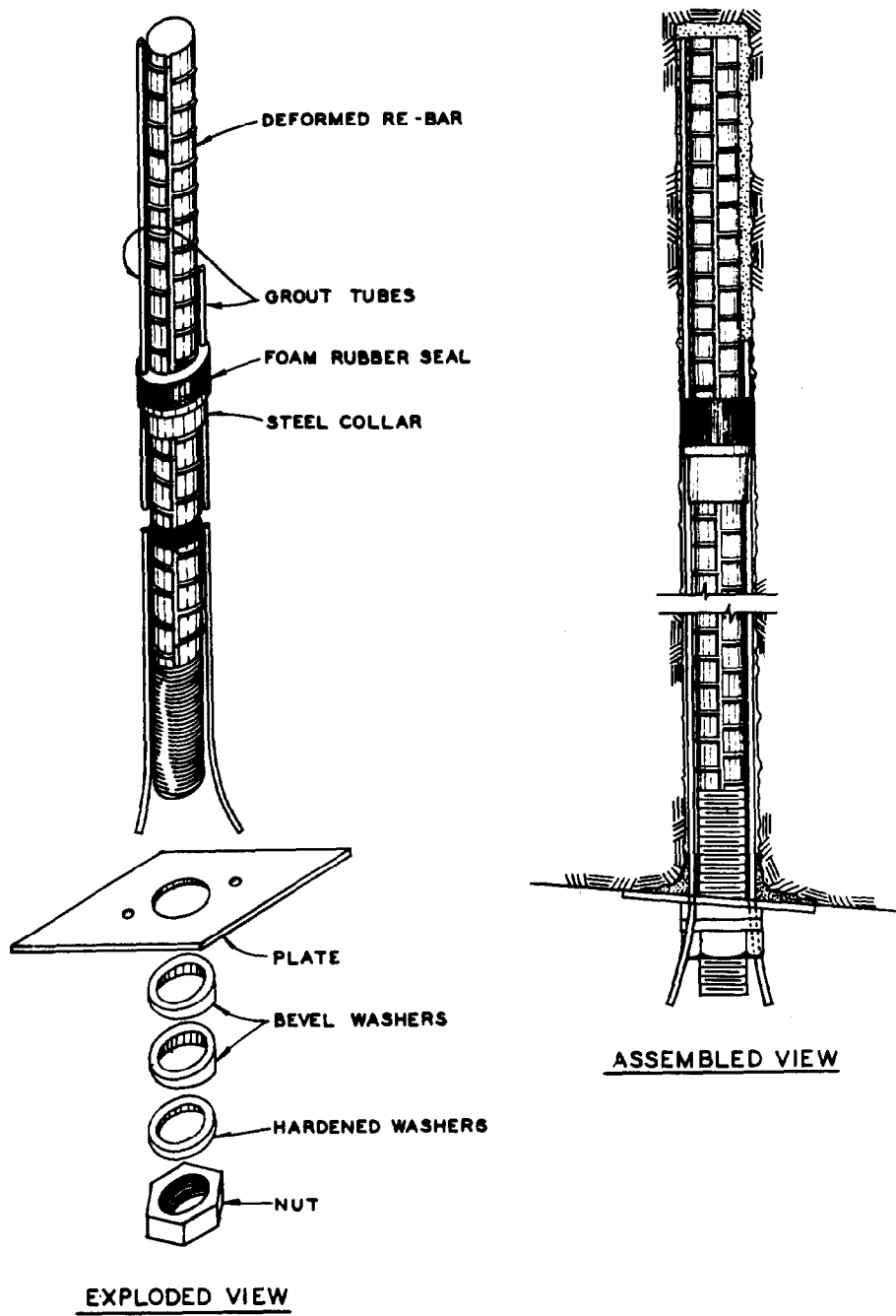
ASSEMBLED VIEW

Figure 3-6. Grouted end anchorage, pumpable type.

BULLET NOSE

PERFORATED SLEEVE
PACKED WITH MORTAR

DEFORMED
RE-BAR

PLATE

BEVEL WASHERS

HARDENED WASHER

NUT

ASSEMBLED VIEW
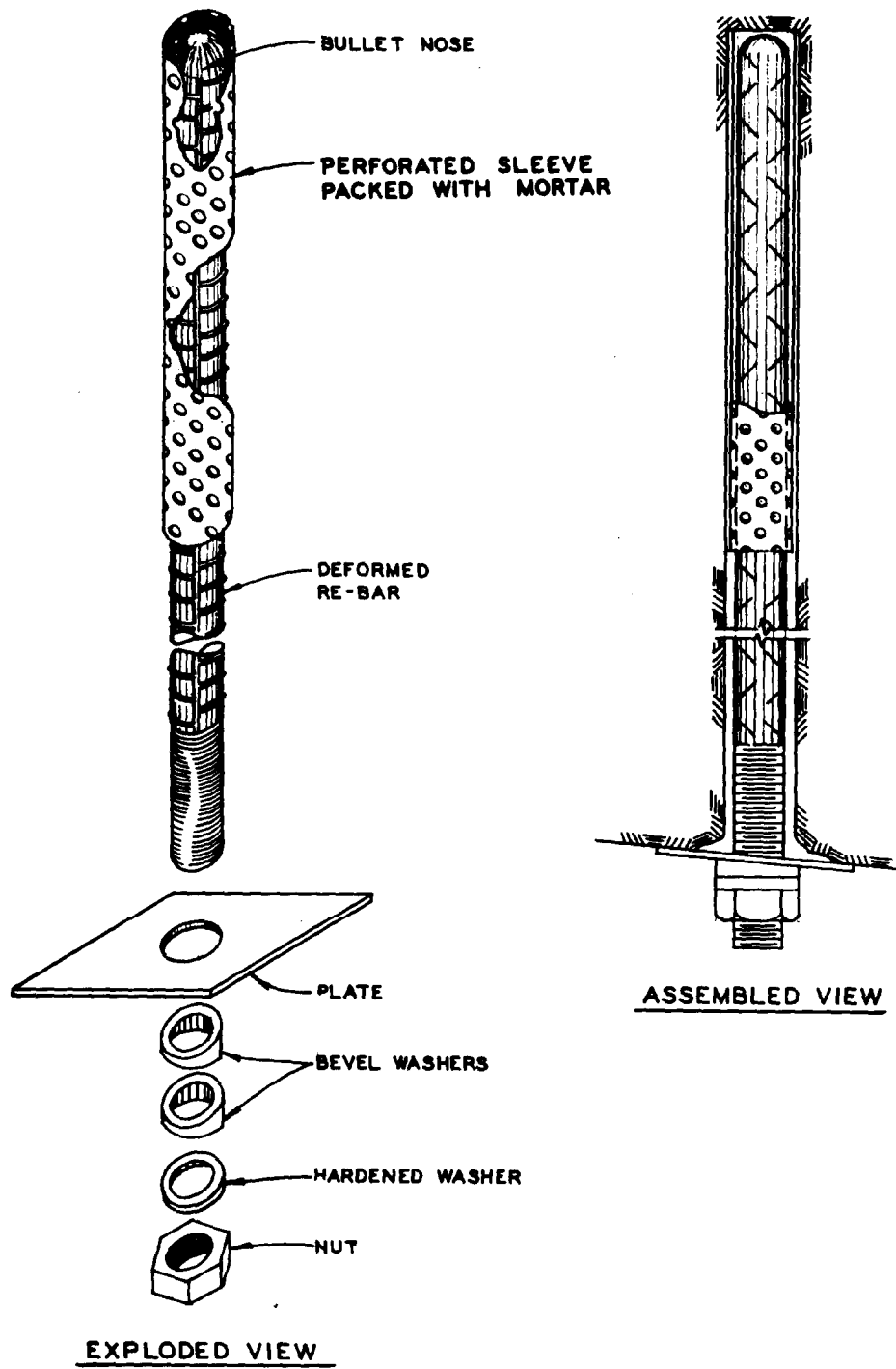
EXPLODED VIEW

Figure 3-7. Grouted end anchorage, perforated
sleeve and mortar type.

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FAST SET RESIN CARTRIDGES FOR
ANCHORING REINFORCING ELEMENT
PRIOR TO TENSIONING

SLOW SET RESIN CARTRIDGES (USE FOR FULL
LENGTH GROUTING)

1. PLACE PREDETERMINED QUANTITY
OF RESIN CARTRIDGES.

LOADING
STICK

MIXED
RESIN

2. INSERT DEFORMED
REINFORCING BAR AND SPIN
THROUGH CARTRIDGES TO MIX
HARDENER AND CATALYST IN CAR-
TRIDGE.

FAST RESIN SETS TO
DEVELOP TENSIONING
ANCHORAGE IN ABOUT
5 MINUTES

STANDARD THREAD

SLOW RESIN SETS IN
20 TO 40 MINUTES

NOTES:
1. "Thread bar",
(see fig. 3-9) may
be used in place of
bar shown.
2. If "thread bar" is used,
only direct pull tensioning is
recommended.

3. PLACE MORTAR BEARING PAD, PLATE,
WASHERS AND UNTIGHTENED NUT.
4. AFTER "FAST" RESIN SETS, TENSION
THE BAR (BY DIRECT PULL OR TORQUING
THE NUT) AND COMPLETE NUT TIGHTEN-
ING OPERATION.
5. AFTER TENSIONING, "SLOW" RESIN
SETS TO PROVIDE CONTINUOUS BOND
OVER FULL LENGTH OF REINFORCING
ELEMENT.
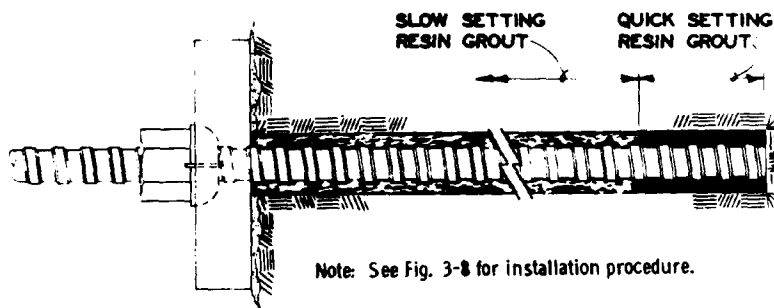
Figure 3-8.   Grouted end anchorage, polyester resin
(includes full-length bonding technique).

CARTRIDGES, POLYESTER
RESIN GROUT

COUNTER-SINK SEAT
FOR LOCKNUT

THREAD BAR

BEARING PLATE

COUPLER FOR
150-KSI BAR
(WHEN REQ'D.)

COUPLER FOR GRADE
60/75 BAR
(WHEN REQ'D.)

LOCKNUT

ROLLED-IN, THREAD-
LIKE DEFORMATIONS,
RIGHT HAND ON
150-KSI BAR.

LEFT HAND THREAD-LIKE
DEFORMATIONS AND NUT ON
GRADE 60/75 BAR. USE WITH
STANDARD ROCK BOLT BEARING PLATE.

EXPLODED VIEW

SLOW SETTING
RESIN GROUT

QUICK SETTING
RESIN GROUT

Note: See Fig. 3-8 for installation procedure.

ASSEMBLED VIEW
(HARDWARE FOR 150-KSI BAR SHOWN)

Figure 3-9. "Thread bar" rock bolt with polyester
resin grouted anchorage.

(6) A variation of grouted anchorage which can be used in soft rocks involves belling the bottom of the drill hole using commercially available belling tools. The bottom end of the reinforcing element is threaded and a plate with a nut on each side is attached. This type of anchor reduces stress concentrations between the anchor and the rock. In this case, anchorage strength is developed by utilizing the bearing capacity of the rock rather than the shear strength at the grout-rock interface.

(7) Table 3-2 lists sources of commercially available grouting equipment, grout retention aids, and materials in common use for bonding reinforcing elements to the rock to form grouted end anchorages. Also listed, are rock bolt systems offered by suppliers which utilize specially constructed reinforcing elements in combination with a bonding medium. Most of the products can also be used to accomplish full length bonding of tensioned rock bolts or untensioned reinforcing elements.

3-3. **Bolts and Accessories.**

a. **General.** Bolts and other components required are listed in the following paragraphs along with a brief discussion of the purpose of each component.

b. **Bolts.** Steel bars used to connect the anchorage to the bearing plate at the collar of the hole are either smooth rods or deformed bars, solid or hollow (groutable), threaded one or both ends, or threaded one end and headed at the other, depending on the type of anchorage and the type of hardware at the collar. Manufacturer's literature should be checked to determine exact size and specified minimum strengths of the bars, since sizes sometimes vary from the nominal sizes given. Specified minimum yield strengths of commonly used bars usually vary between 30,000 psi and 75,000 psi with tensile strengths ranging from 60,000 psi to 100,000 psi. A minimum elongation in 8-inch-gage length of 8 percent is considered acceptable for the high strength steel ranging up to 17 percent minimum for the lower strength steel. Examples of smooth bar rock bolts are shown in figures 3-1 through 3-3. Groutable types, which are manufactured to simplify the task of pumping liquid grout to completely fill the annulus around the rock bolt, are shown in figure 3-4 (smooth bar with integral grout/vent tube) and in figure 3-5 (hollow core deformed bar). Deformed bars are ordinarily used with the types shown in figure 3-6 and 3-7 because a shorter bond length is required than with smooth bars. For the resin anchor types, examples of which are shown in figures 3-8 and 3-9, deformed bars or specially designed smooth bars must be used to achieve good mixing of the resin components. The "thread bar" shown in figure 3-9 is a specially manufactured deformed bar with a continuous rolled-in pattern of threadlike

3-15

Table 3-2.   Some of the Commercially Available Equipment and Materials
for Installing Grouted Reinforcing Elements

Item	Bonding Medium	Type and Size of Reinforcing Element Recommended
Perfo sleeves	Portland cement or gypsum mortar	Smooth or preferably deformed bars, 3/4-in. - 1-3/8-in. diam (No. 6-No.
Moyno grout pump	Neat cement or gypsum grout, chemical grouts	Smooth or deformed bars, all sizes
Williams grout pump	Neat cement or gypsum grout. Chemical grouts	Offered as part of Williams Groutabl Rock bolt system. Can be used similar to Moyno
Sulfa-Set, F-181	Neat gypsum grout or mortar	Use with perfo sleeves or grout pump and elements listed with each
ROC-LOC 540 Mining Kit	Polyester resin. Placed via transfer tube	Any size deformed bar or threaded smooth bar
Celtite System	Polyester resin. Prepackaged in cartridges	Deformed bars, No. 6 through No. 11, No. 14
Dywidag Threadbar Rock Bolt	Celtite resin cartridges	Dywidag high alloy deformed "threadbar." 5/8-in. diam (230 ksi), 1-in. 1-1/4 in., 1-3/8-in. diam (all 150 ksi)
Celtite resin anchor system. Dywidag Threadbar Rock Bolts	Celtite resin cartridges	Deformed bars, No. 6 through No. 11, No. 14. Dywidag Threadbar, 22 mm, Grade 60
FASLOC resin anchored bolt system	Polyester resin prepackaged in cartridges	Specially manufactured deformed bar headed bolt, 3/4-in. diam
Resin-anchor roof bolt	Epoxy resin and stone aggregate in glass cartridges	Specially manufactured smooth bar bolt with threaded ends. 5/8-in. diam - 1-1/4-in. diam

deformations along its entire length. Bars can be cut to the desired length in the field with a portable band saw. The deformations serve as threads to fit specially supplied anchorage nuts or couplings. One supplier offers a rock bolt system which uses a high alloy deformed "thread bar" having an ultimate strength of 150,000 psi. Another supplier offers "thread bar" steel with a yield strength of 60,000 psi. To avoid confusion in the field, the high alloy steel bar is rolled with right-hand thread deformations whereas left-hand deformations are used on the lower strength bars. Direct pull tensioning is recommended with this type of bar. With other resin anchor systems, specially designed deformed bars are sometimes offered and recommended by the suppliers. Types of available bolts and bars are listed in tables 3-1 and 3-2.

c. <u>Bearing Plate and Mortar Pad</u>. Bearing plates are used to spread out and transfer the concentrated bolt load to the rock around the collar of the hole. The bearing capacity of the rock and the prestress load in the elements will govern the size of the bearing plate but 6 inch by 6 inch by 3/8 inch to 8 inch by 8 inch by 3/8 inch for 1-inch bars or 8 inch by 8 inch by 1/2 inch for 1-3/8-inch bars have been found satisfactory in hard rock. If plate deformation is excessive, double plates may be used but increasing plate thickness is better. Some commercial plates have a keyhole slot for passage of grout tubes, or holes may be drilled for grout tubes. The bearing plate should be seated on a pad of quick-setting mortar to provide a uniform bearing surface and to adjust the angle of the plate with the bolt to a more normal position.

d. <u>Bevel Washers</u>. Bevel washers should be used between the bearing plate and the hardened washer to create a uniform bearing surface for the nut normal to the bolt axis. This provides for efficiency in tensioning the bolt by torquing the nut or in transferring the load to the nut when the bolt is tensioned by direct pull. Also, lack of uniform contact between the nut and bearing surface will result in combined stresses in the bolt which will tend to reduce its strength. The bevel angle of washers varies with manufacturer, but angles of 2 degrees, 7 degrees, and 9 degrees are typical examples of those available. By using washers in pairs, the total bevel angle can be varied by rotating one washer relative to the other.

e. <u>Hardened Flat Washer and Thread Lubricant</u>. A hardened flat washer should be used between the bevel washers and the nut in all cases. In addition, the nut bearing surface and the bolt thread should be treated with a molybdenum disulfide base lubricant because this type of lubricant is highly efficient for reducing friction. The hard washer-lubricant combination greatly reduces friction, thereby requiring less torque on the nut to achieve a given tension in the bolt, and also

3-17

reduces torque induced in the bar as the nut is tightened. These considerations are equally important when the bolt is tensioned by direct pull since the nut must still be tightened under load to achieve maximum load transfer from the loading jack to the nut.

f. <u>Nut</u>. The nut should develop the ultimate strength of the bar; generally a heavy duty nut is required. A hexagon nut should be used which is double chamfered or washer faced.

g. <u>Grouting Tubes</u>. Tubes installed in the drill holes for transmitting pumped liquid grout or for venting air should be semirigid. Diameters of tubes which have been used successfully are 3/8-inch outside diameter (OD) and 1/4-inch inside diameter (ID) for cement grout and 1/2-inch OD and 3/8-inch ID for resin grout.

h. <u>Bond Breaker</u>.

(1) A short bond breaker or covering (approximately 6 inches long) over the bolt installed immediately behind the bearing plate is recommended to prevent bond between the bar and the mortar of the bearing pad or grout seal and also to keep the threads clean of mortar. Each of these conditions would tend to reduce the stress in the main part of the bolt relative to the applied stress during the tensioning operation. Lubricants specified for use on threads to reduce friction are also effective as bond breakers. However, to assure better quality control, an additional bond breaker should be specified such as a commercially available waterproof paper mailing tube, several wraps of aluminum foil or even kraft paper fitting snugly around the bolt behind the plate.

(2) Although used infrequently in normal fully grouted rock bolt installations, long bond breakers are useful for distributing working stresses in the steel over a greater length or for making maximum use of the steel ductility. Except for the portion of the bar needed to develop end anchorage, the bond breaker can extend over the full bar length. If a mechanical anchor or plate with two nuts is installed at the anchor end, the bond breaker can be installed full length since forces exerted through the bar on the end anchor will be resisted by the shear force developed along the full area of the grout-rock interface.

i. <u>Couplings</u>. Couplings are available for splicing bolts. These are especially useful when installing bolts from a top heading in a large chamber since bolt lengths required will often exceed the height of the opening. Coupling sizes vary with manufacturer and style and should therefore be checked for compatibility with drill hole size and grouting method used. Outside diameters of standard threaded couplings

are approximately 1.4 to 1.5 times the thread diameter. Couplings for "thread bars," figure 3-9, are larger and outside diameters range from 1.75 to 2.0 times the nominal bar size. All couplings should be designed to exceed the full strength of bolt and in the case of deformed hollow core rebar rock bolts, the couplings should be "stop-type" coupling to ensure the free flowing of the grout through the coupled connection.

j. Miscellaneous. A variety of other accessories such as angle washers, spherical washers, rubber bolt hole sealers, grout tube adapters, plastic washers for holding resin cartridges in place, and impact wrench adapters are available and described in manufacturer's catalogs.

3-4. Installation Methods.

a. General. The success of a tensioned rock bolt installation is largely dependent on the installation techniques employed by the contractor. Since close attention to details is a necessity to attain maximum advantage of the rock bolt system, supervisors with knowledge and appreciation of the basic principles of rock bolting should be employed. Installation methods recommended in the following paragraphs have been developed by trial and error, by laboratory experiments, and primarily by on-the-job experience on several projects. Deviations from these recommendations by inexperienced personnel should be held to a minimum in order to avoid unnecessary problems and the expense of remedial work.

b. Drill Holes.

(1) Close control of hole drilling operations during installation of rock bolts is extremely important for achieving successful rock reinforcement. Hole size, length, condition, location, and alignment are all factors which can significantly affect the installation of particular systems. Hole size is critical for most installations. The length of hole is critical only for slot and wedge and certain grouted and resin anchorages, but for economy reasons, the hole should not be longer than necessary. Irregular, dog-legged, rifled, or undersized holes usually have less serious effects on good anchorage than oversized holes but for maximum bolt effectiveness, these should also be held to a minimum.

(2) Oversized holes, which have a most serious effect on good anchorage, are caused by allowing the bit to spin at the end of the hole, by using incorrectly sharpened bits, by using the wrong size bit, or by using bits which are incorrectly marked as to size. Bit sizes should

always be measured and drill hole sizes checked routinely with a hole measuring gage throughout the construction period, such as the Ohio Brass gage. The standard gage is 10 feet long and measures from 1-1/4 inch to 1-5/8 inch in diameter, but practically any size or diameter will be supplied on special order. The overall hole size range, however, cannot exceed 3/8 inch for each gage. The Williams gage also supplies a drill hole gage which is available in any length with one model for 1-1/2- to 2-inch holes and another model for 2- to 3-1/2-inch holes. The Ohio Brass and Williams devices are both calibrated in 1/16-inch increments but measurements are easily made to 1/32 inch by estimating between markings.

(3) For mechanical anchorages, the hole size should be the smallest diameter that will allow the assembled unit to be pushed into the hole so that a minimum amount of expansion ability is lost. When hole size tolerances are not given by manufacturers, hole diameters should not be more than 1/32 inch larger than the specified diameter and no smaller. For grouted anchorages, except for the pumpable grout types, size is also critical relative to the bar size and quantity of pre-placed grout, but a tolerance of up to 1/16 inch oversize can be allowed.

(4) Some knowledge regarding the capability of rock drills and bits in common use is useful to avoid the possibility of specifying hole diameters and lengths which are difficult or expensive to achieve. Particularly with the harder rocks, the best economy is achieved with the use of percussive or rotary-percussive type drills. The drilling of smaller diameter holes ranging from 1-1/4 inches to 2 inches in diameter is possible with the use of jacklegs utilizing integral drill steel (bit permanently brazed to tip of rod). However, much higher drilling rates are possible with the use of heavier drills, called drifters, mounted on multiple booms on jumbos. With these drills, 1- to 1-1/4-inch-diameter drill steel (rod) is used with detachable bits which drill 1-5/8- to 1-3/4-inch-diameter holes (1-3/4 inches preferable) as a minimum. Drill rods are normally supplied in lengths varying in increments of 2 feet but normally do not exceed 12 feet. For holes longer than 12 feet (or where less headroom exists) extension rods with couplings are required which further limit the minimum size to 1-7/8 inches or 2 inches in diameter. Holes can also be started with a 2-inch diameter or larger bit and finished off in the anchorage area with a smaller bit. This technique works well with mechanical anchorages or grouted end anchorages where subsequent full length bonding is with pumpable grout. However, grouted systems which require or work best with no variation in size throughout the entire hole length will require the use of 1-7/8- or 2-inch-diameter holes as a minimum when installed to depths greater than 12 feet. This, in turn, will influence

the selection of bolt or bar size used to reinforce the rock.

(5)   Drill holes must be cleaned just prior to installation of the bolt to remove sludge, rock dust and particles, and debris present in the hole.   Cleaning can be accomplished by introducing compressed air (minimum of 50 psi) at the bottom of the hole or by washing with water. (In the case of slaking shales, use air only.)   If expansion shell anchorages are being used, down-holes can be overdrilled in length to trap particles, particularly when drilling through high fractured rock.

c.   Mortar Pads.

(1)   Once positive anchorage is achieved and the bolt tensioned, a significant steel stress loss will occur unless a firm bearing surface exists at the rock face to resist the load in the bolt.   Mortar should always be placed under the bearing plate and kept as thin as possible so that high points of the rock surface will remain in contact with the plate to help resist the plate pressure.   The plate should be installed as near normal as possible to the long axis of the bolt.   In this regard, drillers should be instructed to avoid collaring the drill bit in angular recesses or niches in the rock surface when drilling rock bolt holes.   Although advantageous from the driller's standpoint, proper seating of the bearing plate at a workable angle and on a thin mortar pad is made very difficult on the highly irregular rock surface.

(2)   Two parts of quick-setting cement and one part Type III portland cement mixed with sufficient water to form a stiff mix has proven to be a good mortar mix.   The same mix with the addition of two parts sand will also provide good results.   Only the quantity of mortar sufficient for one installation should be mixed at a time.   For proper placement, the mortar is packed in a ball around the bar at the collar of the hole and the bearing plate and nut installed.   In preparation for pressure grouting around the bolt steel, if required, the collar of the hole is also sealed with mortar at this time.   Pressure is then applied to the mortar by rotating the nut until the mortar is evenly distributed under the plate.   Adjustments in the angle of the plate are also possible at this time.   Setting time required varies with condition and angle of the rock surface and the value of the tensioning load.   If the surface is fairly normal to bolt axis and uniformly irregular, and a very thin pad is placed, the bolt can be tensioned in less than a minute just as soon as the mix sets initially.   If a large adjustment in the plate angle is necessary and a thick pad results under a portion of the plate, a setting time of 5 minutes-15 minutes or more may be necessary. Tensioning during the first few minutes following initial set should be avoided since the pad has a tendency to break up during this time.

Setting times to fit different conditions can be quickly determined with a small amount of experimentation.

d. <u>Expansion Anchor</u>. Rock bolt installation should be accomplished immediately after the drill hole is cleaned. The following procedures are for a fully grouted bolt:

(1) The protective sleeve over the shell is removed just prior to insertion.

(2) The threads should be checked by screwing the cone onto the bolt until the threads project through the hole. The cone should spin freely on the threads. If it does not, a small amount of grease should be placed on the threads. Care must be taken to keep grease off the surface of the shell.

(3) Insert assembled rock bolt in the drill hole and set the expansion shell with a calibrated preset impact wrench preferably with an automatic cutoff. Check the correct setting torque with a hand torque wrench periodically. If the rock bolt does not have a hollow core, insert a long plastic tube along the bolt extending to the expansion shell.

(4) Apply coating of molybdenum disulfide base lubricant to threads of bolt.

(5) Install waterproof paper tube bond breaker over threads flush with surface of mortar pad.

(6) Install quick-set mortar around collar of hole to provide a uniform bearing surface for bearing plate. For later pressure grouting operations, position the grout tube or tubes at this time and seal the collar of the hole. Position grout tubes so that weight of bar or shifting of bar will not seal or restrict the tube opening.

(7) Seat bearing plate against mortar before it sets up.

(8) Install bevel washers as necessary to provide uniform bearing surface for hardened washer and nut.

(9) Clean exposed threads and apply thread lubricant over threads and contact surfaces of washer and hex nut.

(10) Install hardened washer and hex nut. Advance nut and position plate on mortar pad as described in paragraph 3-4c.

(11) As soon as sufficient bearing strength is developed, tension

the bolt as described in paragraph 3-5 and grout full length as described in paragraph 3-6.

e. <u>Slot and Wedge Bolt Installation</u>. Slot and wedge bolts should be installed according to the manufacturer's recommendations as modified by the results of the field test program at the site with the following supplementary steps:

(1) The hole must be cleaned as discussed in paragraph 3-4b(5).

(2) The rock bolt (with the wedge inserted in the slot) and the hole depth must both be measured to ensure that adequate length for driving and installing bearing plate, washers, and nut remains. For pressure grouting, a long plastic tube extending to the back of the bolt can be taped in position at this time. Tube should be positioned along slot so pinching of tube will not occur as bolt ears are expanded into rock.

(3) The bolt is driven with an air-powered stoper or drifter until there is no further movement of the bolt into the hole. A driving dolly (available from rock bolt manufacturers) must be used to protect the exposed threads and to keep the bolt from rotating. If preferred, the long grout tube can be inserted along bolt to back of hole at this time rather than as described in step 2.

(4) Continue with installation steps 4 through 11 listed in paragraph 3-4d for the expansion shell type.

f. <u>Grouted End Anchorages</u>. The desirability of using grouted end anchorages over mechanical anchorages increases as rock quality decreases. Once the embedment length for a particular rock type or condition is determined, positive anchorage is possible in 100 percent of all installations. However, experience has shown this to be achievable only with close attention to installation detail. Installation methods and techniques for creating grouted end anchorages are very similar to those used for accomplishing full length bonding of tensioned or untensioned elements. Manufacturer's literature and data sheets as well as practices recommended in this manual should be carefully studied and followed. Field tests must be conducted to determine embedment lengths, to determine required set or cure time of the bonding medium before tensioning of the element is attempted, and for establishing procedures to be used during the construction period. Checks on all installation crews should be conducted routinely throughout the entire construction period for conformity to established procedure.

(1) Pumpable grout type. This type is prepared as shown in

figure 3-6. For full length grouting (after the end grout has set and the bolt has been tensioned) an additional long plastic tube beginning at the threaded end and terminating just short of the packer is also taped to the bar. Polyfoam rubber is usually cut to size and fastened to the bar with a narrow band of tape to form the packer. In up-holes, wooden wedges at the hole collar are used to hold the bar until the grout sets up. Except for providing clearance for the grout tubes and making adjustments to the packer thickness, hole size is not critical and can be almost any size larger than the bar diameter. Normally, a hole diameter equal to the bar diameter plus 5/8 inch to 3/4 inch should be used as a minimum. In softer rocks where bond strength is low at the rock-grout interface, the bonding area can be increased by drilling larger rather than longer holes. Procedures and materials for placing the end anchorage grout are covered in paragraph 3-6. Once the end grout is set, steps 4 through 11 in paragraph 3-4d should be followed to complete the installation.

(2) Perforated sleeve and mortar type.

(a) When installing this type, figure 3-7, the drill hole depth should not exceed the depth to which the reinforcing element will extend into the rock. The size relationship between the drill hole, the reinforcement element, and the perforated sleeve is critical and should be such that the combined cross-sectional area of the bar and the sleeve packed with mortar be 10 percent to 15 percent greater than the area of the drill hole. (Sleeves are of 20-gage metal with perforations over approximately one half the surface area.) Poor anchorage strength may result if a smaller volume of excess mortar is used. A greater volume of excess mortar will make driving of the bar difficult through the mortar. A listing follows which shows the size relationships recommended by the sleeve manufacturer for installing standard deformed bars. Other combinations must be based on separate computations.

Deformed Bar Size	Drill Hole Diam, in.	Perforated Sleeve Diam, in.
No. 6	1-1/4	1-1/16
No. 8	1-1/2	1-1/4
No. 9	1-3/4	1-1/2
No. 10	2	1-3/4
No. 11	2-1/4	2

The following portland cement mortar mix is recommended for packing the sleeves when a setup time of two days or more is acceptable.

Portland Cement Mortar for Packing Perforated Sleeves

Type III portland cement (2 sacks)	188 lb
Sand	188 lb
Admixture (CE Specification CRD-C566)[19]	2 lb

Water: Approximately 0.3 water-cement ratio by weight. Sufficient mixing water should be added to produce a mortar with a flow of approximately 85 percent when tested in accordance with CE Specification CRD-C 116.[16] (Note: A good mix is one that will "pack like a snowball," without exuding free water.)

Shorter setup times are possible with the use of portland cement and accelerators, but four hours or more time is usually required before the element can be tensioned depending on the rock temperature. The quantity of accelerator should be determined on the basis of pull tests conducted at hourly intervals on bars embedded in mortar. A mixture of one part Type I portland cement, one part sand, and sufficient water and accelerator (one part Sika-Set, or equal, to five parts water) to produce a mortar with a flow of 85 percent in accordance with CRD-C 116[16] is recommended as a starter mix for testing. Tests should also demonstrate that the mortar will remain plastic for a sufficient length of time (usually 20 minutes or less) to allow driving of the bar through the mortar. With the use of gypsum cement and sufficient water to form a stiff plastic mix, bars can be tensioned after a setup time of approximately thirty minutes. However, the long term stability (years) characteristics have not been completely investigated. Laboratory tests have shown that when submerged in water, gypsum grout did not increase in bond strength over the seven-day strength. After three months, the water-immersed sample exhibited less than one half the bond strength of a sample cured in dry air.

(b) Installation procedure is identical with that shown in figure 4-1 except that a bar with a threaded end is used. The back end of the bar is ground smoothly to a hemispherical shape to facilitate driving through the mortar. Where holes are not drilled straight, the sleeve may bind in the hole and cause premature extrusion of mortar by the bar. To prevent this, small-diameter copper wire can be laced across the mouth of the sleeve to temporarily restrain the bar until the sleeve reaches the back of the hole. If holes are drilled oversize, immediate correction should be made to the drilling procedure. However, compensation for an oversize hole is possible by overfilling the sleeve to form an elliptical shape when the two halves are tied together.

(c) In preparation for subsequent full length grouting of the element, a long plastic tube stopping short of the final embedment should be taped to the bar before driving or the tube may be inserted just prior to tensioning of the element. Once the end anchorage has developed sufficient strength, the installation is completed by following steps 4 through 11 in paragraph 3-4d.

(3) Polyester resin type.

(a) Resins are available in bulk form or packaged in cartridges as shown in table 3-2. The bulk resin is available in a shipping container which also serves as a mixing container for paste, hardener, and activator packed in separate plastic bags. A polyethelene tube and a plunger are used to load the tube and to transfer the resin to a drill hole prior to inserting a deformed bar through the resin to form the end anchorage. Drill hole size is not critical but should be as small as possible to conserve resin. Resin cure time prior to tensioning the bar varies from 1 hour at 90° F to 24 hours at 45° F. Detailed data sheets are available from the manufacturer which show installation details, resin volume requirements for various deformed bar/hole size combinations, resin gelation and curing times required at different temperatures with varying amounts of activator, and storage instructions. These are not repeated here because this type has not gained wide acceptance for use in civil engineering works. However, the resin has been used successfully in a number of installations, particularly in Canadian metal mines. Special safety precautions are necessary for handling the resin, because it is combustible (flash point 150° F) and component vapors or contact may cause skin and eye irritation.

(b) Methods for installing end anchorages with the use of cartridges will pertain primarily to the types shown in figures 3-8 and 3-9 because a wide range of cartridge diameters with various set times is available. However, installation techniques are similar for all cartridge types listed in table 3-2. Since manufacturers are in the process of expanding existing lines of polyester resin products, the latest data sheets should be obtained from suppliers during the design of rock reinforcement. Steps in the installation sequence and other information are given in figures 3-8 and 3-9. As indicated in the figures, holes are also loaded with sufficient resin to bond the element full length as well as to form the end anchorage. Once the bar has been spun to the end of the drill hole, steps 4 through 11 of paragraph 3-4d are followed after eliminating all reference to grout tubes or full length grouting.

(c) For installing the polyester cartridge types, detailed data sheets are available which show the recommended bar size/hole size/ cartridge size combinations and the respective unit embedment length.

These relationships are critical not only to assure filling the space around the deformed bar but also for other reasons. Shredding of the cartridge is best assured when hole and cartridge diameters are only slightly larger than the bar diameter. With these size relationships, the best mixing of components is achieved and formation of air pockets is prevented because the resin component particles are forced to translate and interact along the bar length. For achieving a good installation an excess resin quantity of 15 percent is allowed. As a general rule, a successful installation will result if the cartridge and drill hole diameters exceed the nominal bar diameter by approximately 1/4 inch and 3/8 inch respectively, i.e., No. 8 rebar, 1-1/4-inch- (32-mm-) diameter cartridge, 1-3/8-inch-diameter hole. This relationship will result in an embedment length of approximately 20 inches for a standard 12-inch-long cartridge. Correct size relationships as well as embedment yields must be positively determined at the time field tests are conducted to establish embedment lengths required for positive anchorage. Over-drilling of hole depth up to 2 inches is acceptable.

(d) Since the amount of resin required is somewhat sensitive to hole size, a problem may occur when longer bolts requiring a coupling are utilized. The hole enlargement required to accommodate the coupling may make the area to be filled with resin grout uneconomical. To remedy this situation, a cement groutable rock bolt has been developed (Fox Industries) incorporating a pressed-on washer which acts as a resin stop at the point of anchorage (figure 3-10). An integral grout tube then permits the remainder of the bolt to be grouted in the usual manner. In this case, the air bleeder tube is terminated just under the resin stop. Bars incorporating these types of features are available in 1-inch, 1-1/4-inch, and 1-3/8-inch sizes.

(e) Cartridges are available in several viscosities and setting times for use at different temperatures or construction conditions. Standard setting times are 1 minute, 2 to 4 minutes, 5 to 10 minutes, and 15 to 20 minutes. Insertion of bar and mixing of resin must be complete within the range of the set time of resin at the end anchorage. The bar can then be tensioned after 5 minutes but tensioning must be complete before the full length bonding resin sets. Because of the rapid set times, hole straightness is critical for rotating the bar without binding in the hole, particularly in long holes. Driving of the bar must not be allowed. Setting times are also sensitive to temperature and steel must be stored to assure temperature compatibility with the resin formulation and viscosity. Checks must also be established to ensure that resin is used within the limit of its storage life.

3-5. Tensioning Methods.

a. General. There are two methods of tensioning and these methods

POLYESTER RESIN
GROUT CARTRIDGE

GROUT/VENT
TUBE

RESIN STOP

TOP VIEW OF
RESIN STOP

PLAIN OR DEFORMED BAR

BEARING PLATE

BEVEL WASHERS

HARDENED WASHER

NUT

EXPLODED VIEW

PRESSED ON
WASHER
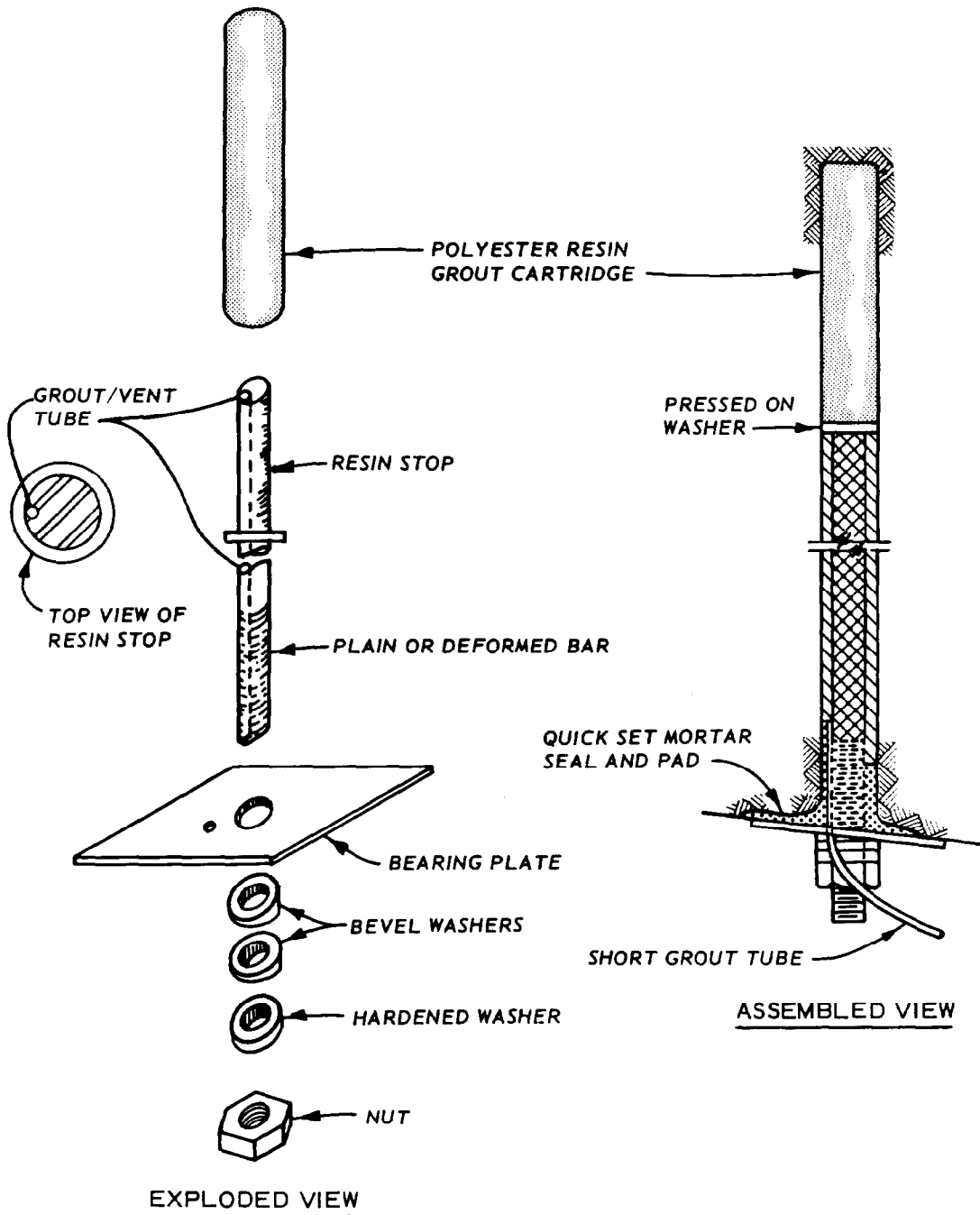
QUICK SET MORTAR
SEAL AND PAD

SHORT GROUT TUBE

ASSEMBLED VIEW

Figure 3-10.   Cement groutable rock bolt with resin
anchor stop and integral grout tube.

are not dependent on the type of anchorage used or the type of bolt used. These are direct pull tensioning using a hydraulic system and torquing of the nut using a torque wrench. Direct pull tensioning has two advantages over torquing. The first is that torsional stresses do not combine with tensile stresses to reduce the strength of the bar. This advantage is not so great if friction reducing materials, as discussed in paragraph 3-3e, are placed prior to torque tensioning. The other advantage is that direct pull tensioning gives a positive indication of the capacity of the anchorage within the range of the tensioning load for every bolt installed. With either method, a deformeter (hollow core rock bolt modified to act as an instrument for monitoring strain in the steel) should occasionally be installed in place of a rock bolt to check the efficiency of the tensioning method. Improvements in the equipment and hardware generally used by contractors with both tensioning methods would be desirable.

b. Direct Pull Tensioning. Installation of the bearing plate as near normal as possible to the long axis of the bolt is a prerequisite to tensioning with presently available equipment. Direct pull tensioning can be accomplished with commercially available rock bolt pullers which are self-contained and attach directly to the rock bolt or by the use of a bridging device along with a bolt extension and a center hole ram with remote hydraulic pump as schematically indicated in figure 3-11. Any commercially available system should be satisfactory but the system should incorporate an easily read dial gage with a large face calibrated throughout the range of the tensioning load on a scale extending at least over a semicircular arc. Jack systems equipped with preset load indicators which give no indication of load prior to or after the preset load is reached are not recommended. Some load loss can be expected when transferring the load from the jack to the bolt nut. In most cases, a large load loss is traceable to insufficient tightening of the nut and controls are necessary to prevent this condition. Using the dial gage as a control, transfer losses can be greatly minimized by tightening the nut until the jack load is reduced by approximately 15 percent prior to releasing the jack load. An alternative method which may be particularly necessary on larger bolts would be to apply a jacking load some 10 or 15 percent larger than design loading to ensure no load loss when transferring the load. Tests should be conducted with the use of a rock bolt deformeter to determine the efficiency of the load transfer operation in the field. The jack system should also be recalibrated frequently during the course of a project to assure accurately recorded tensioning loads. The seating loss for a jack system can also be determined by a lift-off.test. Once this is known, then the bolt can be tensioned to the desired working load plus the load for seating loss.

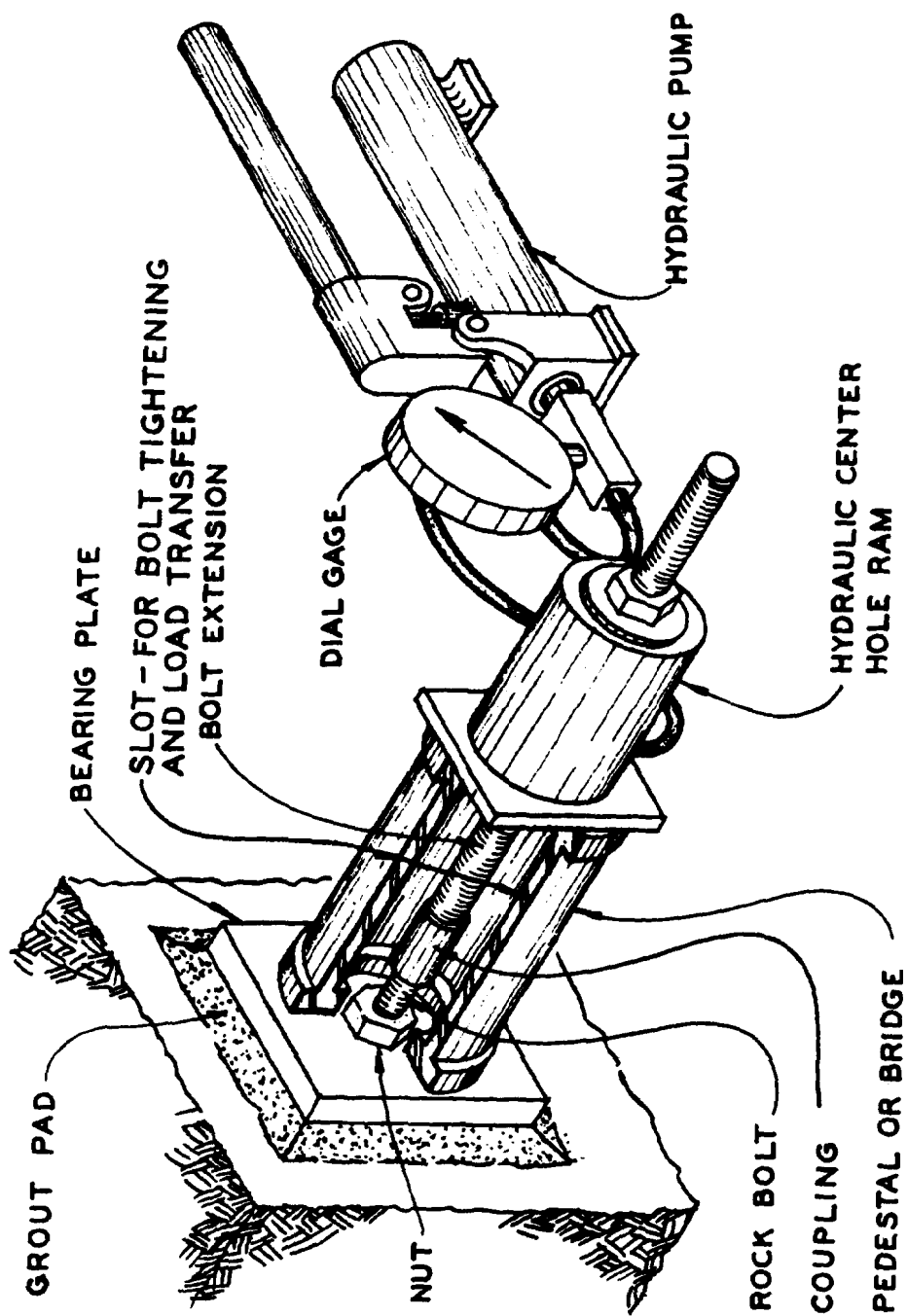c. Torquing. Tensioning by torquing the nut is acceptable

Figure 3-11. Center hole ram for direct tensioning of rock bolts.

particularly when tensioning loads under 30,000 pounds. The installed load is subject to wide variation due to a number of conditions related to the control of alignment and friction between mating parts as well as to size of reinforcing element and undetectable anchor slip. The torque required to produce a specified load is usually expressed empirically as:

$$\text{Bolt tension, lb} = C \times \text{torque, ft-lb}$$

Although C may be defined within narrower limits under controlled laboratory conditions, C can be expected to range from a value of 40 to a value of 80 under actual field conditions, excluding anchor slip and provided installation techniques presented in this chapter are specified. Torquing is usually accomplished with a calibrated air-driven impact wrench. For reliability of results, these required frequent recalibration and a great deal of maintenance. The wrench output is also subject to variations in air line pressure. In all cases, a hand torque wrench must be used as a check on the powered torque wrench. With the hand torque wrench, the nut should be in motion at the time the reading is made. Hand torque wrenches also need to be recalibrated periodically. The manufacturer of a given bolt system will usually provide recommended C-values for his particular product.

    d. Retensioning. Rock bolts should be tensioned and grouted full length prior to continuing blast operations near the bolt installations. If nearby blasting is permitted before grouting, retensioning becomes necessary just before grouting to eliminate any load losses caused by the blasting vibrations. In any event, bolts should be grouted as soon as possible and normally should not be left ungrouted for more than one day.

3-6. Grouting of Reinforcing Elements. Installation methods and techniques for creating grouted end anchorages or for accomplishing full length grouting after tensioning of elements are very similar. In both cases, successful installations will result by following recommended procedures with close attention to all of the installation details. Manufacturer's literature and data sheets should also be consulted.

    a. Pumpable Liquid Grout. Liquid grout is usually used for the anchorage shown in figure 3-7 and for full length bonding of the types shown in figures 3-1 through 3-7. Grout should always be injected at the lowest point in the hole so that air will escape as the grout level builds up. With hollow core bolts installed in up-holes, the hollow core is used for venting and a short length of plastic tube is grouted in the collar of the hole for injecting the grout. The process is reversed for down-holes. For the type in figure 3-6, a similar procedure is followed for creating the end anchorage except that the long tube

substitutes for the hollow core. For full length grouting of the types shown in figures 3-6 and 3-7, a long tube terminating at the anchorage must be taped in place prior to placing the reinforcing element and a short tube installed at the collar. The same two-tube technique is used for full length grouting of types shown in figures 3-1 through 3-3. With these types, the long tube is best placed after the bolt is anchored to avoid crimping the tube as the bolt is spun. A long thin removable rigid rod or tube inserted in the plastic tube will simplify the placement. This method is particularly useful when it becomes necessary to upgrade older installations where ungrouted bolts were installed.

(1) Portland cement grout. Although various bonding materials are possible, the most common is neat portland cement grout. The following mix is recommended for forming grouted end anchorages where a fast setup time (less than 2 days) is not required or for full length grouting of elements previously tensioned.

Type III portland cement (2 sacks)	188 lb
Flyash (optional) - check for possible alkali reactivity	75 lb
Admixture (CE Specification CRD-C566)[19] Grout Fluidifier and Expanding Agent	2.6 lb
Water: Approximately 0.4 water-cement by weight. Quantity of water sufficient to produce a grout efflux time of 20 seconds when tested in accordance with CE Specification CRD-C79-58.[15]	

For pumping neat cement grout, all grout pipes, tubes, and fittings should be free of dirt, grease, hardened grout, or other contamination before grouting commences. All wash water and diluted grout should be flushed from the lines. The grout line should be attached to the injection point such that leakage is entirely prevented. As a general rule, grout pressure at the collar should not exceed 25 psi and grouting should be continued until there is a full return through the vent. To avoid clogging the tubes, the grout must be passed through a No. 8 screen prior to injection because the cement will sometimes "ball-up" to form obstructions. If, during grouting, grout leaks to the surface through open joints and appreciable grout is being lost, the grouting operation should be temporarily suspended and the joints caulked with quick-setting mortar or other caulking material. If, during grouting of any bolt, the hole accepts more grout than required to fill the nominal volume of the

annular space without return of grout through the vent tube and if no leakage is visible at the surface, then grouting operations should be temporarily suspended, the grout line disconnected, and the bolt hole allowed to drain. No earlier than one hour and not later than two hours after suspension of grouting, the grout lines should be reconnected and grouting completed. If excess leakage still occurs, sand should be added to the grout to stiffen it. When grout flows in a steady stream from the vent tube, the vent tube should be plugged (with a golf tee or other plug) while pressure is maintained on the injection tube. The grout line should then be removed and the injection tube plugged.

(2) Polymer and gypsum grout. Epoxy or polyester grout may be substituted for neat cement grout for pumping around the reinforcing element. These have not been used extensively because specialized equipment is necessary which has not been fully developed for economical operation. Gypsum grout can also be pumped using one part gypsum to three to five parts water. However, as mentioned previously, the long term stability of gypsum has not been completely investigated.

b. Packaged Polyester Grout. Installation techniques for accomplishing full length bonding with cartridges are covered in paragraph 3-4f(3).

3-7. Anchorage Capability.

a. As emphasized throughout this manual, determination of anchorage capability in a particular rock type must finally be determined by conducting field pull tests at the site. Figure 3-12 illustrates equipment commonly used to perform a pull test and monitor the behavior of a system as the load is applied. When making direct pull tests of rock bolt it is recommended in underground and bedded formations that safety props be installed to protect the man conducting the test.

b. For making the initial choice of anchorage (prior to the time a site is available for making tests), past experience is useful for selecting alternate systems or for making preliminary estimates of mechanical or grouted end anchorage capabilities. Manufacturer's data sheets also provide some guidance.

c. Table 3-3 summarizes results of anchorage tests conducted at various projects. Work in this area is only partially complete and additional information needs to be gathered and summarized.

3-8. Quality Control. The quality control program for a rock reinforcement system should begin with a test installation at the project
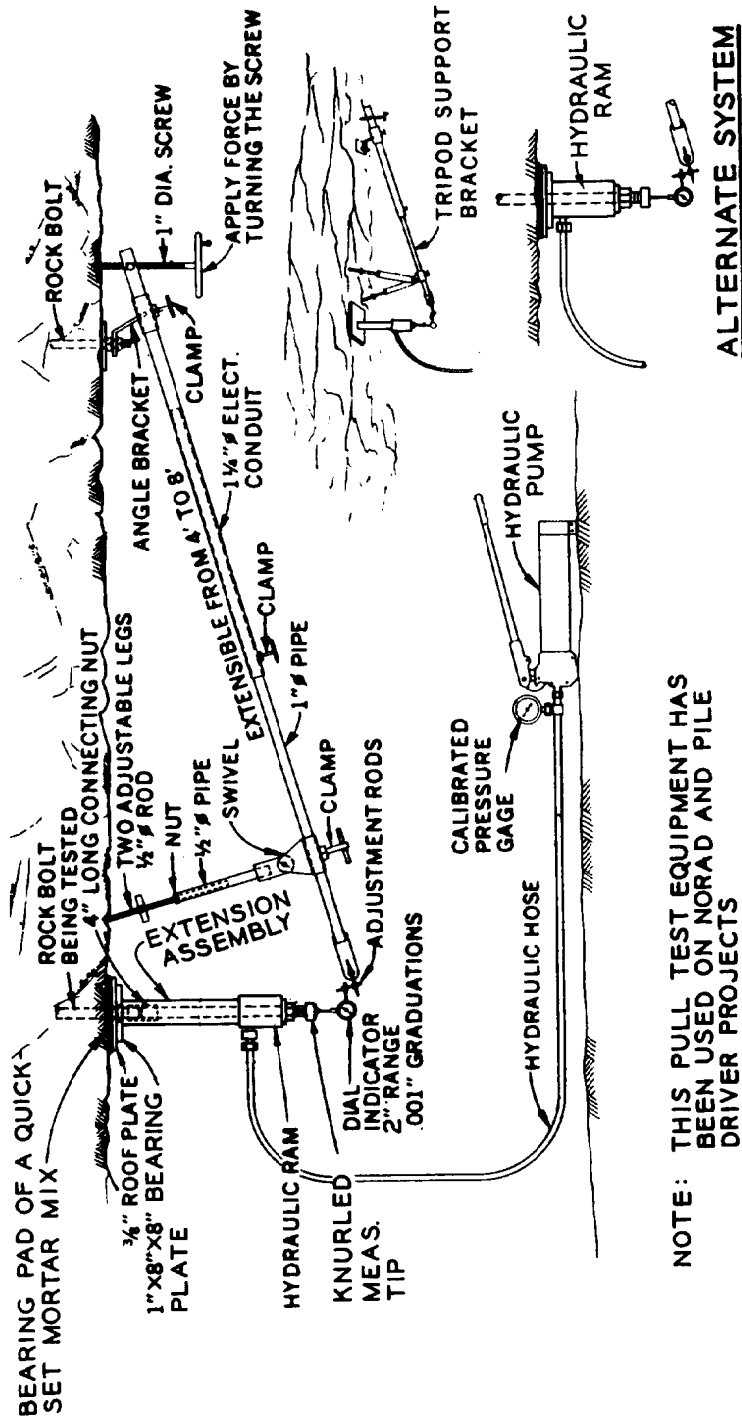
Figure 3-12. Pull test equipment.

Table 3-3. Summary Sheet - Rock Bolt Pull Tests

Project Location	Testing Agency	Rock Type and Description	General Comments
Nevada Test Site	Omaha District, Corps of Engineers	Quartz Monzonite, light gray, dense, porphyritic with a fine- to medium-grained ground mass. G_s = 2.67; q_u = 30,500 psi and tensile strength = 1,450 psi. Rock was competent for all but 3 bolts tested.	A quick setting mortar pad was used under all bearing plates. No. 8 and No. 11 bolts were hollow-core. Testing was performed in 1965.

Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load kips Anchor Failure	Failure Load kips Bolt Failure	Remarks
14 Smooth		170	6	Williams Shell	750	None	--	--	--	--	--	--	--	170	
14 Smooth		170	6	Williams Shell	750	None		--	--	--	--	--	--	175	
14 Smooth		170	6	Williams Shell	750	Neat Cement			--	--	4	34 days	--	160+	Loading discontinued. No failure
14 Smooth		170	6	Williams Shell	0	Neat Cement			--	--	4.5	34 days	--	160+	Loading discontinued. No failure
11 Deformed	71	118	16	Williams Shell	400	None	--	--	--	--	--	--	72	--	Manufacturer's rating is 74 kips at yield and 100 kips at ultimate. Lab tests showed 71 and 118 kips, respectively, through bolt and 59 and 98 through threads
11 Deformed	71	118	16	Williams Shell	400	None	--	--	--	--	--	--	84	--	→
11 Deformed	71	118	16	Williams Shell	400	None	--	--	--	--	--	--	--	62	
11 Deformed	71	118	6	Williams Shell	400	None	--	--	--	--	--	--	36	--	
11 Deformed	71	118	6	Williams Shell	400	None	--	--	--	--	--	--	82	--	
11 Deformed	71	118	6	Williams Shell	400	None	--	--	--	--	--	--	76	--	→
11 Deformed	71	118	6	None	--	Cem-Sand & Perfo Shell	1:1	0.29 Max.			4	28 days	--	96	Grout mix was 188 lb of cement, 188 lb sand, 56 lb water (max), and 2 lb fluidifier
11 Deformed	71	118	6	None	--	Cem-Sand & Perfo Shell	1:1	0.29			3.66+	28 days	--	92	
11 Deformed	71	118	8	None	--	Cem-Sand & Perfo Shell	1:1	0.29			7.75	28 days	--	92	
11 Deformed	71	118	6	None	--	Gypsum (S-1) Perfo					4	1.1 hr	--	90	
11 Deformed	71	118	6	None	--	Gypsum (S-1) Perfo					3	2 hr	--	94	
11 Deformed	71	118	6	None	--	Gypsum (S-1) Perfo					2	2 hr	--	92	
11 Deformed	71	118	6	None	--	Gypsum (S-1) Perfo					1	3 hr	84		
11 Deformed	71	118	8	Williams Shell	0	Neat Cement		0.4	10:36	23:00	5	28 days	--	96	Grout mix was 188 lb of Type III cement, 75 lb flyash, and 2.6 lb fluidifier. Water was added to produce efflux time of 20 sec
11 Deformed	71	118	16	Williams Shell	0	Neat Cement		0.4	10:36	23:00	4	28 days	--	92	
8 Deformed	38	59	8	Williams Shell	0	Gypsum (S-1) (pumpable)					6.5	1.5 hr	--	40	

(Continued)

Table 3-3. (Continued)

Project Location				Testing Agency		Rock Type and Description								Failure Load, kips		General Comments
														Anchor Failure	Bolt Failure	
Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period				Remarks
																Nevada Test Site (Continued) — Test equipment included 10,000-psi capacity hydraulic hand pump, 60-ton and 100-ton center pull rams, extension assembly, and support bracket for dial gage used to measure bolt displacement during testing.
11 Deformed	71	118	8	Williams Shell	0	Neat Cement		0.4	10:36	23:00	4	28 days		--	90	
11 Deformed	71	118	8	Williams Shell	0	Neat Cement		0.4	10:36	23:00	4	28 days		--	92	
11 Deformed	71	118	6	Williams Shell	0	Neat Cement		0.4	10:36	23:00	3	28 days		--	90	
11 Deformed	71	118	16	Williams Shell	0	Epoxy					5	30 days		--	84	A packer was used to control grouted length and grout was pumped in
11 Deformed	71	118	8	Williams Shell	0	Epoxy					5	30 days		--	80+	Loading discontinued. No failure
11 Deformed	71	118	8	Williams Shell	0	Epoxy					4	30 days		--	90	
11 Deformed	71	118	8	Williams Shell	0	Epoxy					3	30 days		--	94	
11 Deformed	71	118	8	Williams Shell	0	Epoxy					1.5	30 days		50	--	
11 Deformed	71	118	8	None	--	Polyester (P-1)					4	7 days		--	92	Resin inserted in back of hole and bolt is driven through it. A transfer tube is used to place the resin
11 Deformed	71	118	8	None	--	Polyester (P-1)					3	7 days		40	--	
11 Deformed	71	118	8	Williams Shell	0	Polyester (P-1)					3	7 days		25	0	
11 Deformed	71	118	8	Williams Shell	0	--					2	7 days		1	--	
11 Deformed	71	118	8	None	--	Polyester (P-2)					4	7 days		--	100	P-2 polyester is furnished in sausage-shaped plastic bags which are placed at back of hole. Bolt is driven through and rotated to mix
11 Deformed	71	118	6	None	--	Polyester (P-2)					3	8 days		65	--	
11 Deformed	71	118	8	None	--	Polyester (P-2)					2	7 days		32	--	
8 Deformed	38	59	6	Williams Shell	250	None					--	--		38	--	Manufacturer rates yield at 37 kips and ult at 50%. Lab tests indicated 38 and 59 for bolt and 28 and 44% through threads. Badly fractured rock
8 Deformed	38	59	6	Williams Shell	250	None					--	--		14	--	Rock was badly fractured
8 Deformed	38	59	6	Williams Shell	250	None					--	--		2	--	Rock was badly fractured
8 Deformed	38	59	8	Williams Shell	0	Gypsum (S-1) pumpable					6.5	2 hr		30	--	

(Continued)

(Sheet 2 of 9)

Table 3-3. (Continued)

	Project Location	Testing Agency		Rock Type and Description										General Comments	

Nevada Test Site (Continued)

Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load, kips — Anchor Failure	Failure Load, kips — Bolt Failure	Remarks
8 Deformed	38	59	8	Williams Shell	250	Gypsum (S-1) pumpable	--	--	--	--	6.5	4 hr	--	40	
8 Deformed	30	59	8	Williams Shell Xtra-Long	250	None	--	--	--	--	--	--	0	--	Improperly installed
7 Deformed			4	C.F.I. Shell	200	None								44	
7 Deformed			4	C.F.I. Shell	200	None							8		
7 Deformed			4	C.F.I. Shell	200	None							4		Improperly installed. Hole drilled 1/8 in. oversize
7 Deformed			4	C.F.I. Shell	200	None								40	
7 Deformed			4	Williams Shell	200	None							8		
7 Deformed			4	Williams Shell	200	None							10		
7 Deformed			4	Williams Shell	200	None							6		
7 Deformed			6	None		Gypsum (S-1) Perfo					4	1 hr	37		
7 Deformed			6	None		Gypsum (S-1) Perfo					4	2 hr		40	

General Comments:

For additional information about this series of tests, refer to "Investigation of Rock Bolt Anchorages in Quartz Monzonite Rock," pile-driver project, Technical Report No. 1, July 1965, Omaha District.

Key

Table Name	Manufacturer's Name	Manufacturer
Epoxy	Epoxy, Formulation 62EP	George W. Whitesides Co. Louisville, Ky.
Polyester (P-1) Polyester (P-2)	ROC-LOC 60 ROC-LOC 20	American Cyanamide Co. Stanford, Conn.
Gypsum (S-1)	F-181 Bolt Anchor Sulfaset	Ranco Industrial Products Corp., Cleveland, Ohio
Rock Bolts	Hollow Core Rock Bolts	Williams Form Eng. Co. Grand Rapids, Mich.
Perfo	Perfo Sleeves	Sika Chemical Co. Passaic, N.J.

(Continued)

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(Sheet 3 of 9)

Table 3-3. (Continued)

Project Location	Testing Agency	Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Rock Type and Description	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	General Comments	Failure Load, kips Anchor Failure	Failure Load, kips Bolt Failure	Remarks
Stockton Dam, Missouri	Kansas City District Corps of Engineers							Limestone								All bolts tested in this series of tests were Williams High Strength Hollow Groutable, Type US-H-HC. The bolt has a nongrouted rating of 47 kips, maximum working load of 74 kips, and ultimate load of 100 kips.			
		11	74	100	9.4	None	--		Sand-Cement (Perfo)					4	3 days		55 (no failure)	--	Grout for perfo shells was 1 part sand, 2 parts Type I cement and enough solution of 3 parts water to 1 part Sika-Set to make mortar workable
		11	74	100	10	None	--		Sand-Cement (Perfo)					4	6 hr 7 min		70.3	--	No failure but spreading of threads froze the nut
		11	74	100	10	None	--		Sand-Cement (Perfo)					4	7 hr 5 min		70.3	--	No failure but spreading of threads froze the nut
		11	74	100	10	None	--		Sand-Cement (Perfo)					4	7 hr 19 min		73.1	--	No failure but spreading of threads froze the nut
		11	74	100	10	None	--		Sand-Cement (Perfo)					4	23 hr 10 min		73.8	--	No failure but spreading of threads froze the nut
		11	74	100	10	None	--		Sand-Cement (Perfo)					2	18 hr		56.4	--	No failure and no displacement
		11	74	100	10	None	--		Sand-Cement (Perfo)					3.8	6 hr		56.4	--	No failure and no displacement
		11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300		None					--	--		51	--	No failure. Testing stopped at 0.78 in. displacement
		11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300		None					--	--		51	--	No failure. Testing stopped when displacement reached 0.95 in.
		11	74	100	10	Williams Short (1-5/8 in.) Cone Type	300		None					--	--		52.5	--	No failure. Displacement reached 0.8 in.
		11	74	100	10	Williams Short (1-5/8 in.) Cone Type	800		None					--	--		82	--	No failure. Threads were spread out at 82 kips. Displacement was 1.4 in.
		11	74	100	20	Williams Short (1-5/8 in.) Cone Type	700		None					--	--		45.2	--	No failure. Displacement was 0.55 in. when testing discontinued
		11	74	100	20	Williams Short (1-5/8 in.) Cone Type	700		None					--	--		39	--	Complete failure 30 sec after reaching 39 kips
		11	74	100	10	Williams Short (1-5/8 in.) Cone Type	700		None					--	--		36	--	Complete failure in 45 sec

(Continued)

(Sheet 4 of 9)

Table 3-3. (Continued)

Project Location			Testing Agency			Rock Type and Description							General Comments		

Stockton Dam, Missouri (Continued)

Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load kips Anchor Failure	Bolt Failure	Remarks
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	700	None					--	--	41	--	No failure
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	500	None					--	--	41	--	Displacement exceeded 1.6 in.
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	500	None					--	--	46.7	--	No failure. Displacement was 0.9 in.
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None					--	--	46.7		
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	300	None							41.2	--	No failure. 0.7-in. displacement
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None							41.2	--	No failure. 1.7-in. displacement

(Continued)

Table 3-3. (Continued)

	Project Location	Testing Agency					Rock Type and Description							General Comments		
Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load, kips Anchor Failure	Bolt Failure	Remarks
Stockton Dam, Missouri (Continued)													Reference for this series of tests: "Rock Bolt Anchor Design," Supplemental Design Memorandum No. 7B, Stockton Dam and Reservoir, Kansas City District, Corps of Engineers, February 1966.		
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None							50	--	No failure. Early stress loss
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None							40	--	Complete failure in 3 min
11	74	100	20	Williams Short (1-5/8 in.) Cone Type	300	None							50	--	No failure
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	300	None							28.7	--	Complete failure in 1-1/2 min
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	300	None							50	--	No failure
11	74	100	10	Williams Short (1-5/8 in.) Cone Type	300	None							46.7	--	No failure
11	74	100	11	Williams Long (3 in.) Cone Type	500	None							55	--	No failure
11	74	100	11	Williams Long (3 in.) Cone Type	880	None							69	--	No failure
11	74	100	11	Williams Long (3 in.) Cone Type	800	None							59.2	--	No failure
11	74	100	11	Williams Long (3 in.) Cone Type	800	None							59.2	--	No failure
11	74	100	21	Williams Long (3 in.) Cone Type	800	None							59.2	--	No failure
11	74	100	21	Williams Long (3 in.) Cone Type	560	None							45.2	--	No failure

(Continued)

Table 3-3. (Continued)

Project Location							Rock Type and Description							General Comments	
Nevada Test Site							Alternating layers of red and white tuff. The tuff is commonly pumiceons and zeolitized.							This test program was conducted at NTS by Fenix and Scisson in cooperation with Reynolds Electrical Engineering Company.	
	Testing Agency														
	Fenix and Scisson, Inc.														
Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load, kips Anchor Failure	Failure Load, kips Bolt Failure	Remarks
1-in.-diam		83	16	None	--	Sulfaset (pumped)					7		--	83 (coupling failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	16	None	--	Sulfaset (pumped)					7		--	74 (thread failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	16	None	--	Sulfaset (pumped)					7		10 (poorly grouted)	--	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7		0 (poorly grouted)	--	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7		53 (poorly grouted)	--	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7		--	82 (thread failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7			73 (coupling failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7		--	57 (coupling failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	32	None	--	Sulfaset (pumped)					7		--	63 (no failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	40	None	--	Sulfaset (pumped)					7		10 (poorly grouted)	--	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	40	None	--	Sulfaset (pumped)					7		--	72 (coupling failure)	Bolt was smooth and anchorage was 8-ft-long "Pigtail" grouted in
1-in.-diam		83	12	None	--	Sulfaset in perfo sleeve					1		0	--	Bolt was smooth, A431 steel
1-in.-diam		83	12	None	--	Sulfaset in perfo sleeve					5		10	--	Bolt was smooth, A431 steel
8		72	12	None	--	Sulfaset in perfo sleeve					2		53	--	Bolt was No. 8 rebar, A431, NC threads
8		72	12	None	--	Sulfaset in perfo sleeve					4		--	72	Bolt was No. 8 rebar, A431, NC threads
9	80	96	16	None	--	Sulfaset (pumped)					7		--	94	Bolt was No. 9 rebar, A431, NC threads

(Continued)

Table 3-3. (Continued)

| Project Location | | | | Testing Agency | | | Rock Type and Description | | | | | | | General Comments | | |
| Nevada Test Site (Continued) | | | | | | | | | | | | | | | | |
Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load, kips Anchor Failure	Bolt Failure	Remarks
9	80	96	24	None	--	Sulfaset (pumped)					5		10	--	Bolt was No. 9 rebar, A431, NC threads
9	80	96	24	None	--	Sulfaset (pumped)					5		81	--	Bolt was No. 9 rebar, A431, NC threads
9	80	96	32	None	--	Sulfaset (pumped)					7		63	--	Bolt was No. 9 rebar, A431, NC threads
9	80	96	32	None	--	Sulfaset (pumped)					7		87	--	Bolt was No. 9 rebar, A431, NC threads

(Continued)

Table 3-3. (Concluded)

Project Location				Testing Agency			Rock Type and Description							General Comments		
Bolt Size No.	Yield Strength kips	Ultimate Strength kips	Bolt Length ft	Mechanical Anchorage Type	Setting Torque ft-lb	Grout Type	Cement-Sand Ratio	Water-Cement Ratio	Initial Set min:sec	Final Set min:sec	Grouted Length ft	Grout Setting Period	Failure Load, kips Anchor Failure	Bolt Failure	Remarks
Nevada Test Site (Continued)													Results of tests are from a report by E. J. Cording, F. D. Patton, and D. U. Deere, "Rock Bolt Tests in U12g Tunnel, Nevada Test Site," Fenix and Scisson, Inc., July 1965.		
9	80	96	32	None	—	Sulfaset (pumped)					9		—	96	Bolt was No. 9 rebar, A431, NC threads
11	57	70	12	None	—	Sulfaset (pumped)					7		—	69	Bolt was No. 11 rebar, A431, NF threads (1-1/8 in.)
11	57	70	12	None	—	Sulfaset (Perfo)					4		—	72	Bolt was No. 11 rebar, A431, NF threads (1-1/8 in.)
11	57	70	12	None	—	Sulfaset (Perfo)					5		35	—	Bolt was No. 11 rebar, A431, NF threads (1-1/8 in.)

site before the contract is advertised. The purpose of this initial
program is to familiarize field personnel with installation procedures
and to assure the designers that the system selected meets design cri-
teria. Another test program should be included as part of the contract.
The purpose of this test is to train contractor's personnel in correct
installation procedures and impress upon them the importance of atten-
tion to details. If direct pull tensioning is used during construction,
the anchorage of every bolt is verified. If torquing is specified for
tensioning, a small percentage (0.5 percent is customary) of the ele-
ments should be selected at random and tensioned by direct pull as a
check on the anchorage. Immediate full length grouting of elements will
eliminate requirements for checking for tension loss. Otherwise peri-
odic checks with the use of a torque wrench will be required until
grouting is completed. Special rock bolt load cells are available
for measuring stress in bolts over lengthy periods and for measuring
restrained relaxation of rock strata. During grouting, an inspector
should be detailed to observe and ensure that specified procedures are
followed and that all elements are fully grouted.

3-9. Specifications. Samples embodying most of the concepts and pro-
cedures recommended in this manual are included as Appendices B and
C. Some minor revisions or additions to these are anticipated.

3-10. Sample Problem to Demonstrate a Method of Underground Rock Anchor
Reinforcement.

    a. Background.

    (1) Despite extensive theoretical and model studies of the behav-
ior of a single bolt or anchor when embedded in rock or of the behavior
of a system of such reinforcing elements, the practical design of rock
reinforcement continues to involve application of empirical rules,
tempered by the experience, judgment, and observations of the designer.
Studies show that when an underground opening is made in rock, deforma-
tion occurs in the near vicinity of the opening creating a loosened
zone with compressive stresses being concentrated further back within
the rock mass creating a supportive ground arch. Timely and proper
installation of rock reinforcement restrains the loosened rock and pre-
vents further loosening at greater distances that could lead to local
or general fallout of rock. The depth of the loosened zone depends
on several factors such as geologic conditions, size of opening with
respect to spacing of rock joints, shape of opening, orientation of
opening with respect to orientation of rock joints, groundwater con-
ditions, construction procedure, and the engineering properties of the
rock mass, particularly the rock mass strength. Although these factors

are important to varying degrees, current Corps of Engineers guidance (EM 1110-2-2901[3]) emphasizes the size of the rock block to be supported, the size of the opening, and overburden rock load pressure.

(2) Although progress is being made in more completely understanding rock reinforcement/rock interactive behavior, it is unlikely that the empirical approach will ever be completely replaced. The inability to predict such important factors as geologic conditions and engineering properties of the rock mass will undoubtedly foster the continued use of empirical rules which, in turn, will require the designer to exercise considerable judgment. This requirement does not particularly detract from the use of rock reinforcements for underground support, especially when consideration is given to the adaptability of the reinforcement to meet unforeseen rock conditions or special construction procedures.

(3) The following example illustrates the application of current CE guidance in the design of rock reinforcement of an underground opening by the use of tensioned rock anchors.

b. Description of Problem. A 10-foot-diameter circular tunnel is to be constructed in a jointed granitic rock formation. Geologic investigation has shown the rock mass to contain two closely spaced conjugate joint sets (figure 3-13). One joint set has an average joint spacing of 12 inches, while the intersecting joint set has an average joint spacing of 18 inches; the strike of the joint set parallels the tunnel axis. From experience or observation it is assumed that the depth of the loosened zone will be on the order of 2 feet.

c. Analysis.

(1) Table 3-4 lists the current design guidance and shows the stepwise procedure to determine minimum length, maximum spacing, and minimum average confining pressure for rock reinforcement. The spacing of the intersecting joint sets and their resulting orientation with respect to the tunnel axis has been determined to yield critical and potentially unstable blocks in the crown of the tunnel (figure 3-13) with dimensions of approximately 2 feet. From the assumed behavior of the rock, no significant loosening is anticipated below the spring line. Therefore, the empirical rules are not used to determine required rock reinforcement below the spring line. In weaker rocks some loosening could occur and support, as determined in the field, might be required.

(2) In trial 1, the minimum length of rock reinforcement is

3-45

Table 3-4. Determination of Minimum Length, Maximum Spacing, and Minimum Average Confining Pressure for Rock Reinforcement (Reference Tables 3-7 and 3-8; EM 1110-2-2901[3])

Parameter · Empirical Rules	Trial 1	Trial 2
Minimum length greatest of:		
a. Two times the bolt spacing	Undetermined	2 × 3 = 6 ft ←Use
b. Three times the width of critical and potentially unstable blocks	3 × 2 = 6 ft	As before
c. For elements above the spring line:		
1. Spans less than 20 ft; one-half span	0.5 × 10 = 5 ft	As before
2. Spans from 20 to 60 ft; interpolate between 10- to 15-ft lengths, respectively	NA	As before
3. Spans from 60 to 100 ft; one-fourth span	NA	As before
d. For elements below the spring line:		
1. For openings less than 60 ft high, use lengths determined in c. (above)	NR	As before
2. For openings greater than 60 ft high; one-fifth the height	NR	As before
Maximum spacing least of:		
a. One-half the bolt length	0.5 × 6 = 3 ft	As before
b. One-and-one-half times the width of critical and potentially unstable rock blocks	1-1/2 × 2 = 3 ft	As before
c. Six feet	6 ft	As before
Minimum average confining pressure at yield point of elements greatest of:		
a. For elements above spring line:		
1. Pressure equal to a vertical rock load of 0.20 times width of opening	0.2 × 10 × 170/144 = 2.4 psi	As before
2. Six pounds per square inch	6 psi	As before
b. For elements below spring line:		
1. Pressure equal to a vertical rock load of 0.10 times the opening height	NR	As before
2. Six pounds per square inch	NR	As before
c. For elements at intersections, twice the greatest confining pressure determined in a. or b. (above)	NR	As before

Note: NA, not applicable.
NR, not required.

JOINT SET A
AVG SPACING 12 IN.

JOINT SET B
AVG SPACING 18 IN.

ESTIMATED LIMIT
OF LOOSENED ZONE *

6-FT ROCK ANCHOR

3 FT

3 FT

3 FT

3 FT

3 FT

3 FT

10 FT

SPRING LINE

JOINTED ROCK, MEDIUM

LOOSE BLOCKS LIKELY TO FALL OUT
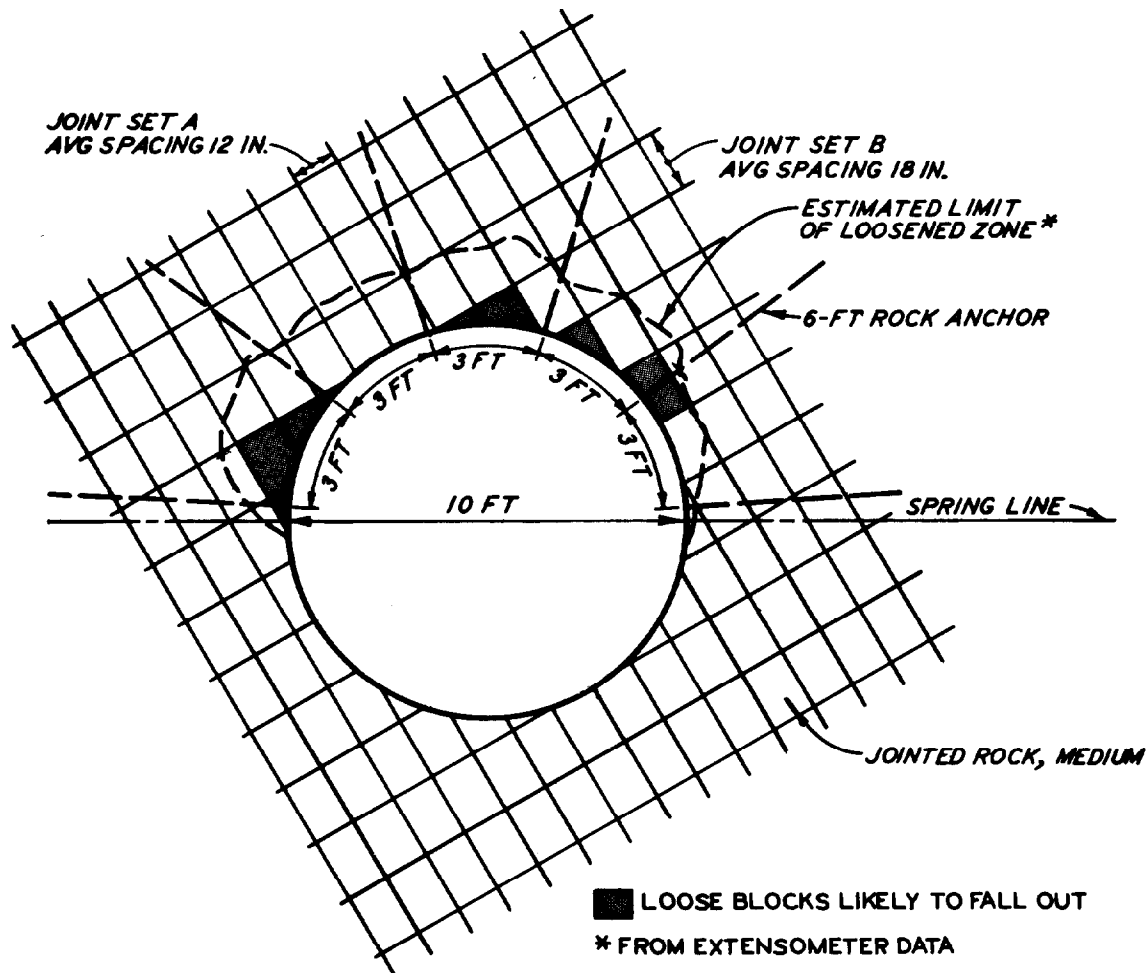
* FROM EXTENSOMETER DATA

Figure 3-13. Example of underground reinforcement.

tentatively shown to be 6 feet with a maximum spacing of 3 feet.
Assuming the unit weight of granite to be 170 pounds per cubic foot
(pcf), then as shown in table 3-4, the minimum confining pressure at
yield point of the reinforcing elements should be 6 pounds per square
inch (psi). The final selection of parameters is determined from
trial 2.

(3) From the selected parameters listed in table 3-4, the tension
in each bolt should be sufficient to cause an average confining pressure
on the tunnel surface of 6 psi. Since the selected anchor spacing is
3 feet on centers, each anchor must support 3 × 3 = 9 square feet
(1296 square inches). Therefore, each anchor must be stressed to a
minimum load of 1296 × 6 = 7800 pounds. Since any anchor size selected
can easily be tensioned to several times this minimum load and still be
below the yield strength, the tensile force in the rock anchor is not
a critical factor in the anchor selection. Guidance is given in
EM 1110-2-2901;[3] tables 3-5 and 3-6, on commercially available rock bolts
with mechanical anchorage and grouted reinforcing elements.

3-11. **Sample Problem to Demonstrate a Method of Surface Slope Reinforcement by Rock Anchors.**

a. **Description of Problem.** Figure 3-14 shows a planned intake
channel for pumped storage units. The 1000-foot-long channel will be
blasted in sedimentary rock with bedding planes dipping from right to
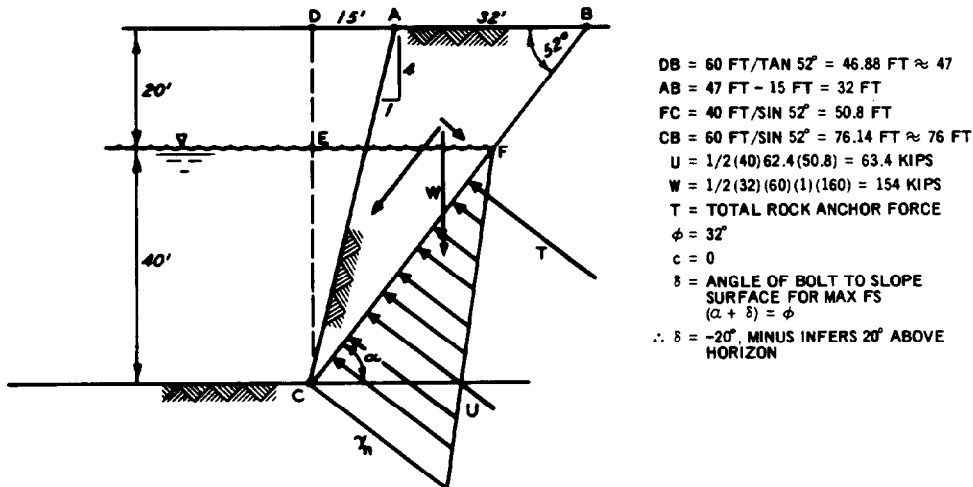

$DB = 60 \text{ FT}/\text{TAN } 52° = 46.88 \text{ FT} \approx 47$
$AB = 47 \text{ FT} - 15 \text{ FT} = 32 \text{ FT}$
$FC = 40 \text{ FT}/\text{SIN } 52° = 50.8 \text{ FT}$
$CB = 60 \text{ FT}/\text{SIN } 52° = 76.14 \text{ FT} \approx 76 \text{ FT}$
$U = 1/2(40)62.4(50.8) = 63.4 \text{ KIPS}$
$W = 1/2(32)(60)(1)(160) = 154 \text{ KIPS}$
$T = \text{TOTAL ROCK ANCHOR FORCE}$
$\phi = 32°$
$c = 0$
$\delta = \text{ANGLE OF BOLT TO SLOPE SURFACE FOR MAX FS}$
$(\alpha + \delta) = \phi$
$\therefore \delta = -20°, \text{ MINUS INFERS } 20° \text{ ABOVE HORIZON}$

Figure 3-14. Example problem (planned intake channel for
pumped storage units).

left at 52 degrees as shown. Presplit side slopes at 1H on 4V are planned to minimize rock breakage on the surface and to minimize disturbance at depth along weak bedding planes. From a cursory inspection it appears the right bank may be unstable since bedding planes dipping into the excavation daylight along the right bank. Accordingly, a rock bolt stabilization scheme will be designed to reinforce the dipping strata in the right bank cut.

    b. _Analysis._

    (1) From inspection it is obvious that the bedding plane which intersects the toe of the slope (BC, figure 3-14) is the most critical when considering the stability of the right bank. Therefore, it is proposed to design a rock bolt system to overcome the forces tending to promote sliding on BC or outward movement from hydrostatic pressure. The 40-foot-deep pool shown can be raised or lowered in a matter of hours due to fluctuating peak power demands. Because of characteristically low joint and bedding plane permeabilities, it is assumed that full pool hydrostatic pressure will exist in both banks during rapid pool drawdowns.

    (2) From an inspection of the sketch and from calculations, it is obvious that excavating the intake channel to bottom grade without incremental rock bolt reinforcement would result in a natural slope adjustment at least from AC to BC during blasting on the right bank. For this reason, it is necessary to excavate the channel in lifts, utilizing full rock bolt reinforcement on the cut slopes at each excavation stage. If no adjacent structures are located critically near the right bank, the economics of cutting the right bank channel parallel to the natural bedding planes (BC in sketch) should be considered. In this case, the rock bolt reinforcing could be substantially reduced. However, in this example problem some 36,000 cubic yards of additional rock excavation would be required. Since the bedding planes in the left bank dip advantageously into the cut slope, this bank can probably be stabilized by using only short rock bolts intended mainly to prevent surface ravelling.

    (3) Because the intake channel leads directly to the forebay of power generating units, it is often prudent (for erosion control or hydraulic requirements) to provide permanent protection to the rock surfaces in the sides and bottom of the cut. For these cases, either a thin dowelled concrete facing or a wire-mesh-shotcrete design might be specified, each constructed with appropriate drain relief holes drilled through the concrete and into rock. Where extensive ravelling of the slope is anticipated or the slope height needs to be reduced

for stability, benching may be required. Bench width, also calculated for stability to prevent subtoe or failure beneath the bench, should be adequate for access by maintenance equipment, with the bench height determined from stability calculations.

(4) The following procedures should generally be followed to determine the rock bolt stabilization scheme:

(a) Calculate factor of safety (FS) of the unreinforced slope using equation 1 below to determine the need for rock bolts. The FS should be calculated for both the wet and dry conditions.

(b) Decide upon the minimum FS necessary to insure slope stability. Generally, a lower FS can be tolerated for temporary slopes or for slopes that are continuously monitored by instrumentation. Note that the contribution of the rock bolt shear strength to the slope stability is ignored in determining the FS.

(c) Calculate the total rock bolt force T (equation 2) required for a given FS to stabilize a 1-foot length of the cut. Decide upon bolt strength.

(d) From T determine rock bolt spacing.

(e) The rock bolt lengths should, in general, be determined such that when using the design bolt spacing, the force cone generated from the bottom of the anchor overlaps the cone from adjacent anchors when projected to the discontinuity surface. In the example problem however, the geometry is such that this is not a practical approach. Past experience and a good knowledge of the slope geology are used to determine bolt length. The bolt should penetrate the plane of the critical discontinuity to a depth that insures anchorage in sound rock. It is usually prudent to stagger the bolt depths so as to avoid defining a weak plane at the anchor tips. When using resin-type grouts for bolt anchors, the manufacturer's recommendations for length of resin column should be followed. This, in part, helps determine the length of bolt. When deciding upon an anchor system, pull-out tests are advisable to verify anchorage capacity. The following equation[28] should be used in calculating the FS:

$$FS = \frac{1}{W \sin (\alpha + \varepsilon)} \{cH \operatorname{cosec} \alpha + [W \cos (\alpha + \varepsilon)$$

$$- u + T \sin (\alpha + \delta)] \tan \phi + T \cos (\alpha + \delta)\} \quad (1)$$

where

W = weight of rock

δ = angle of rock bolt with horizontal; bolt sloping up above horizontal is of a negative angle

α = angle of discontinuity to the horizontal

ε = angle of resultant force due to earthquake or blasting forces and W

c = cohesion of rock

H = vertical distance from point where discontinuity daylights on slope to top of slope

U = groundwater force

T = total force due to rock anchors
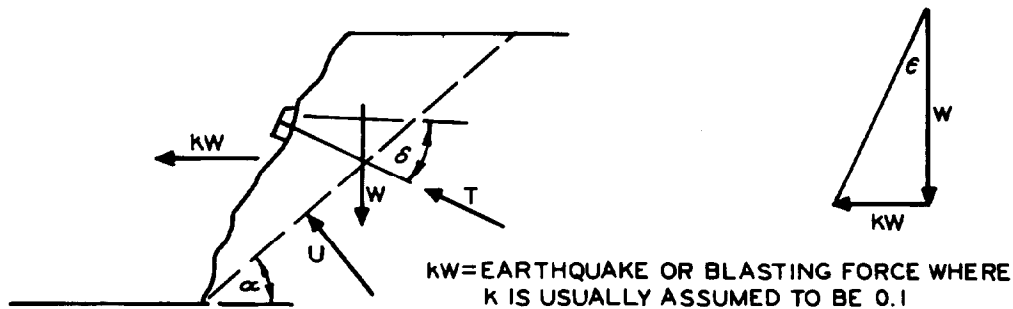
The force polygon for earthquake or blasting force is shown in figure 3-15.

kW=EARTHQUAKE OR BLASTING FORCE WHERE
k IS USUALLY ASSUMED TO BE 0.1

Figure 3-15. Force polygon for earthquake or blasting force.

Equation 1 was selected for the calculation of the FS since the equation contains most of the parameters required for slope-stability analysis and is straightforward to use. The existence of other equations and techniques, which define FS differently, are recognized; however, their use changes the numerical value of the FS only and not the slope stability. To determine the optimum direction of the rock bolts, equation 1 is differentiated with respect to δ . This gives a maximum FS when

$$\tan (\alpha + \delta) = \tan \phi$$

c. Solution.

(1) Step 1. Check for stability:

Dry Static Case

Assumptions: T = 0; U = 0; earthquake and blasting effects ignored ($\epsilon = \delta$); $\alpha + \delta = \phi$

From equation 1

$$FS = \frac{1}{154 \sin 52°} \; (154 \cos 52°) \tan 32°$$

$$FS = 0.49$$

Since the calculated FS is less than 1.0 there is no need to check for hydrostatic case. The slope is unstable, therefore, excavate and bolt in stages.

(2) Step 2-3. Calculate T .

For this example T will be calculated for FS = 1.0 , 1.1, and 1.2. Solve equation 1 for T . This gives:

For FS = 1.0

$$T = \frac{FS[W \sin(\alpha + \epsilon)] - cH \, \text{cosec} \, \alpha - W \cos (\alpha + \epsilon) \tan \phi + U \tan \phi}{\cos (\alpha + \delta) + \sin (\alpha + \delta) \tan \phi} \quad (2)$$

For FS = 1; $(\alpha + \delta) = \phi$; $\epsilon = 0$; U = 63.4 kips

$$T = \frac{1(154) \sin 52° - 154 \cos 52° \tan 32° + 63.4 \tan 32°}{\cos 32° + \sin 32° \tan 32°}$$

$$T = 86.2 \text{ kips/foot of cut length}$$

(3) Step 4. Calculate bolt spacing.

Use a bolt working capacity of 102 kips and a discontinuity surface area of 76 sq ft (CB × 1). The bolt working capacity was assumed for this problem.

Required bolt force/sq ft = $\dfrac{86.2 \text{ kips}}{76 \text{ sq ft}}$ = 1.13 kips/sq ft

∴ each bolt will strengthen $\dfrac{102 \text{ kips}}{1.13 \text{ kips/sq ft}}$ = 90.27 sq ft

S_B = bolt spacing = $\sqrt{90.27 \text{ sq ft}}$

S_B = 9.5 feet

Bolt should be placed on 9.5-foot centers on the plane of the discontinuity inclined at an angle of -20° (52° + δ = 32° ∴ δ = -20°) for FS = 1.

For FS = 1.1

$$T = \frac{1.1(154) \sin 52° - 154 \cos 52° \tan 32° + 63.4 \tan 32°}{\cos 32° + \sin 32° \tan 32°}$$

T = 96.5 kips/foot of cut length

Required bolt force/sq ft = $\dfrac{96.5 \text{ kips}}{76}$ = 1.27 kips/sq ft

∴ each bolt will strengthen $\dfrac{102}{1.27}$ = 80.32 sq ft

S_B = $\sqrt{80.32}$ = 8.96 feet

S_B = 9.0 feet

Bolt should be placed on 9-foot centers on the plane of the discontinuity inclined at an angle of -20° for FS = 1.1.

For FS = 1.2

$$T = \frac{1.2(154) \sin 52° - 154 \cos 52° \tan 32° + 63.4 \tan 32°}{\cos 32° + \sin 32° \tan 32°}$$

T = 106.8 kip/ft of cut length

Required bolt force/sq ft = $\frac{106.8}{76}$ 1.41 kips/sq ft

∴ each bolt will strengthen $\frac{102}{1.40}$ = 72.86 sq ft

S_B = $\sqrt{72.86}$ = 8.53 feet

S_B = 8.5 feet

Bolts should be placed on 9-foot centers, on the plane of the discontinuity inclined at angle of -20° for FS = 1.2. The required bolt spacing on the plane of the cut slope is determined from figure 3-16 using the law of sines.
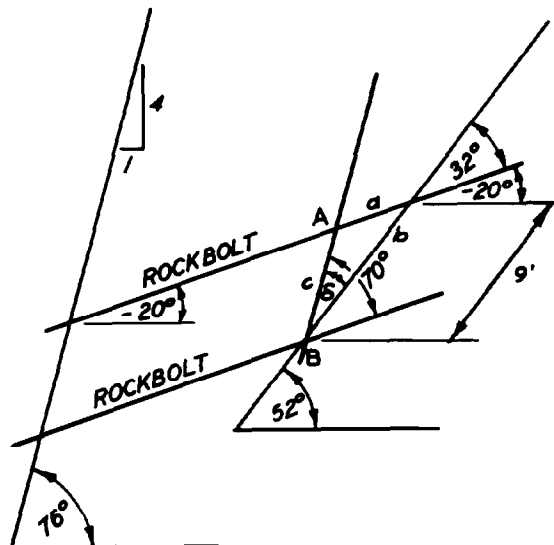

Figure 3-16. Calculation of actual bolt
spacing for FS = 1.1.

Construct line AB parallel to cut slope and intersecting discontinuity at point c.

γ = 56° – δ = 52° – 20°   α = 76° – 52°   B = 180° – (α + γ)

γ = 32°   α = 24°   B = 180° – (56°) = 124°

From law of sines:

$$c = \frac{b \sin \gamma}{\sin B} = \frac{9 \sin 32°}{\sin 124°}$$

$$c = 5.75 \text{ feet}$$

∴ actual bolt pattern on cut surface should be 6 feet vertical by 9 feet horizontal for FS = 1.1.

    d. Summary.

Figure 3-17 shows the relationship between the angle of inclination of the bolt and the bolt force required for a given FS for the example problem. The curves demonstrate that for small changes in the inclination of the bolt (in the range from $\delta = -20°$ to $\delta = 0°$), the FS would not change significantly. As an example, changing the bolt angle from $\delta = 20°$ to $\delta = 0°$ and using T as calculated for $\delta = -20°$, the FS changes from 1.1 to 1.04 (from equation 1). Installation of the bolt at $\delta = -20°$ may be impractical; if so, and if a $\delta = 0°$ is desired, the bolt force and spacing would be calculated as outlined above. Bolt inclinations below the horizontal ($\delta > 0°$) should be avoided. The bolt length can be determined graphically from figure 3-17 adding a suitable length beyond the plane of the discontinuity to satisfy anchorage requirements.
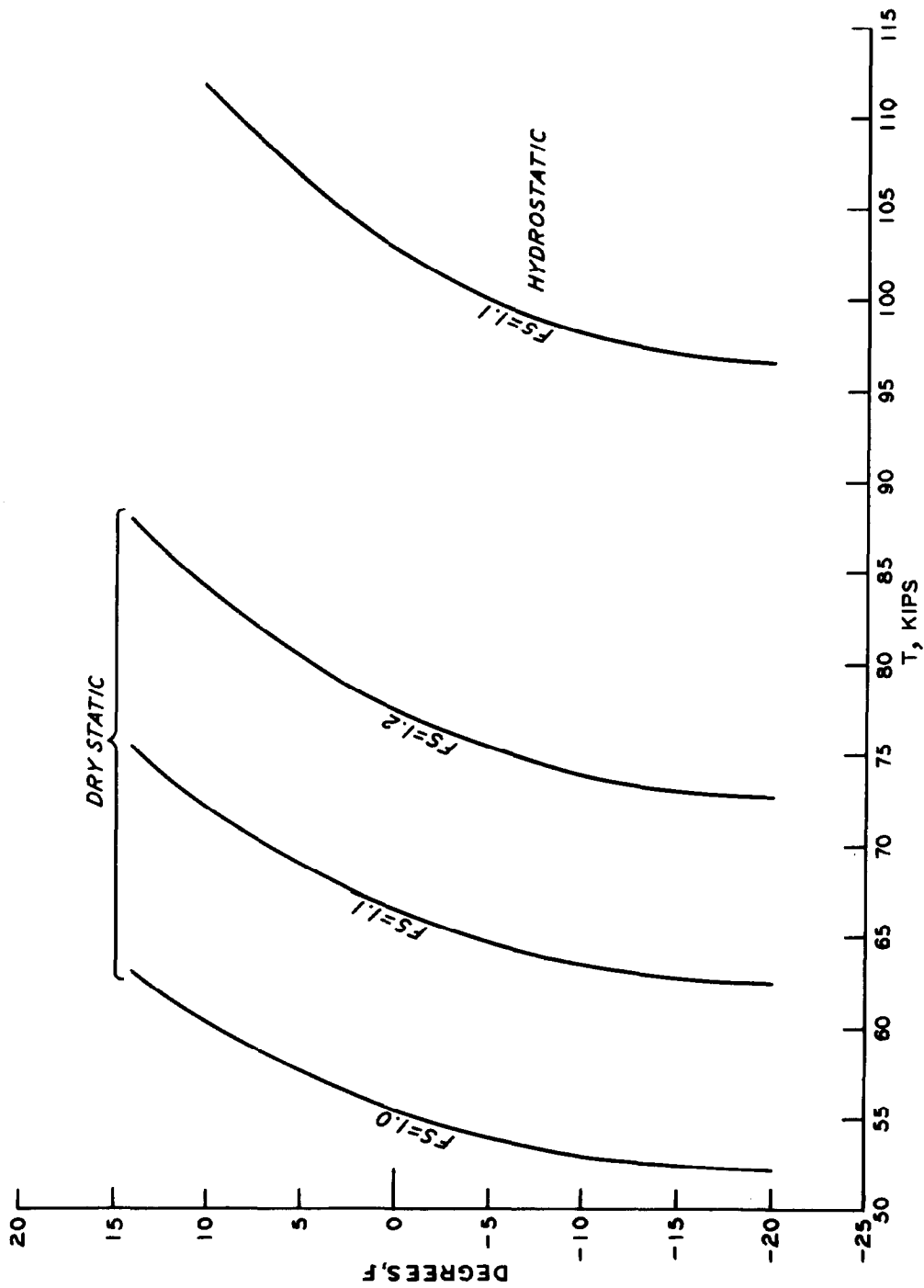
Figure 3-17. Relationship between angle of inclination and bolt force for a given FS in example problem.